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ACCELERATION LANES FOR TURNING VEHICLES AT RURAL INTERSECTIONS

FINAL REPORT

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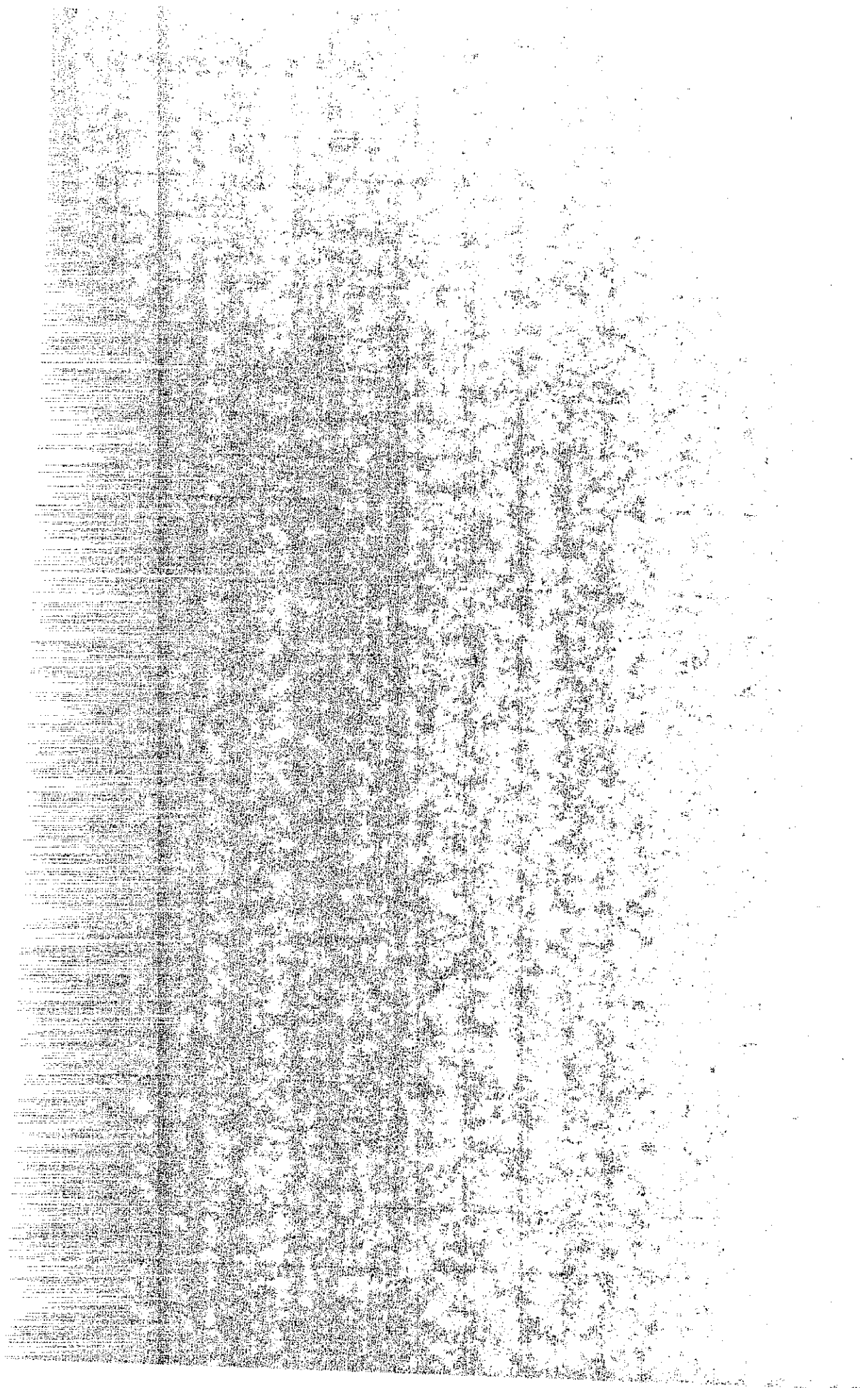
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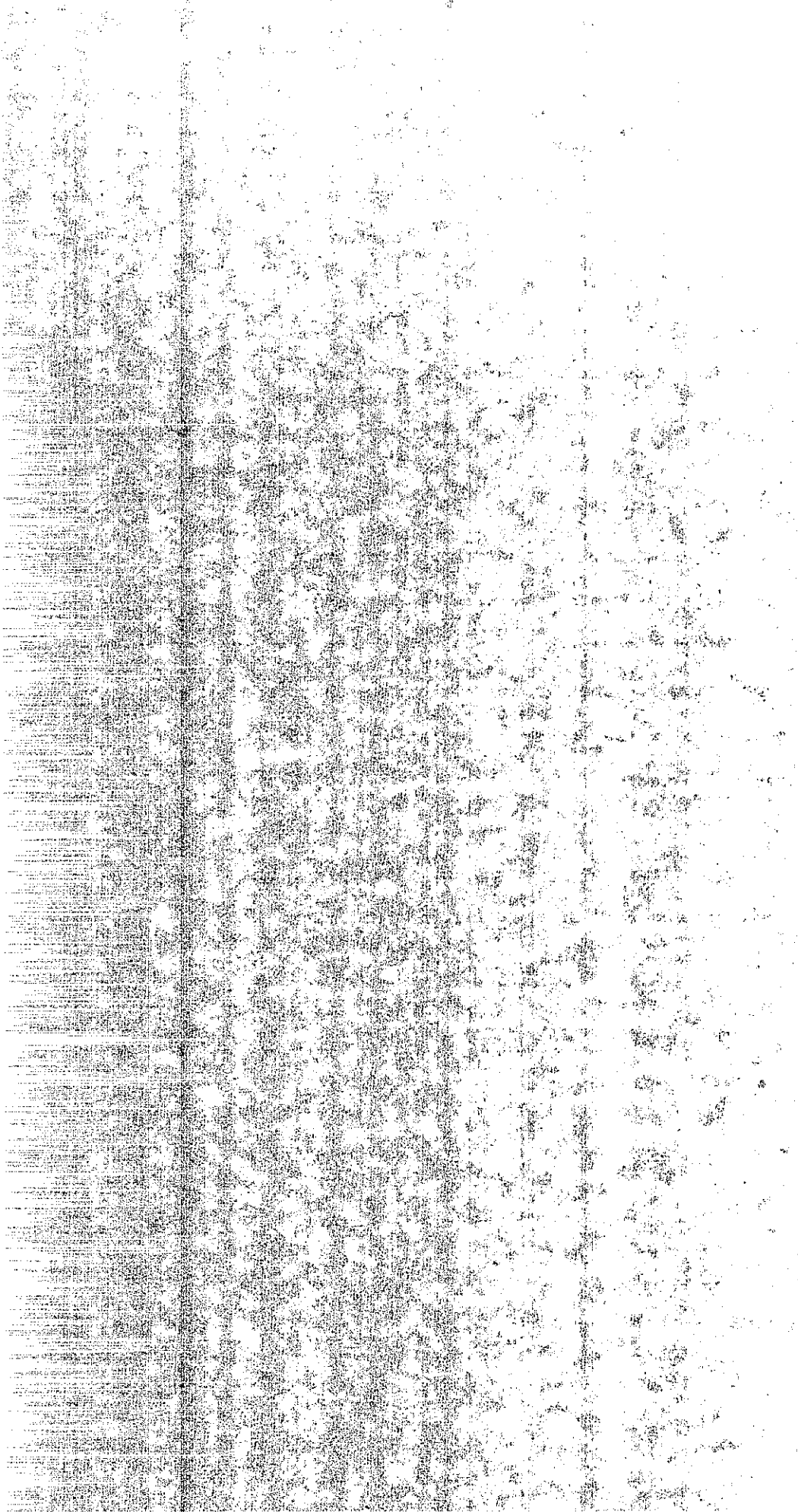


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The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data and information contained herein. The contents do not necessarily reflect the official views or policies of the State of California or of the Federal Highway Administration. The report does not constitute a standard, specification or regulation.



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1. INTRODUCTION

The California Highway Design Manual (1) does not discuss acceleration lanes for vehicles turning either left or right onto four-lane or two-lane highways. Some Caltrans districts do, however, provide acceleration lanes. A research study was initiated "to determine if acceleration lanes are needed for vehicles turning left or right from cross roads onto both four-lane and two-lane highways" (2). Part of the justification was to "be consistent in the application of acceleration lanes so as to minimize exposure to tort liability".

It would appear that there is a need for guidelines for acceleration lanes as well as for guidelines for the appropriate dimensions of the acceleration lanes. From discussions with Caltrans personnel it was concluded that the types of intersections where the greatest need for guidelines currently exists, are rural four-leg intersections, with stop control on two-lane cross roads, and where high speeds prevail on the major road.

Based on the above, two specific objectives for this study were formulated:

- (a) To determine guidelines for implementing acceleration lanes for right- and left-turning traffic onto four-lane or two-lane rural high-speed highways at four-leg stop-controlled intersections.

- (b) To determine the appropriate dimensions of the acceleration lanes.

In order to achieve the objectives, a survey of existing practice as well as an operational and a safety analysis was carried out.

The overall approach to the research is outlined in Chapter 2. A survey of existing practice is presented in Chapter 3. The operational analysis is described in Chapter 4 and the safety analysis is presented in Chapter 5. Chapter 6 contains a summary of major conclusions. Guidelines for implementation are discussed in Chapter 7 and recommendations are presented in Chapter 8. Examples of intersection layouts, obtained from New Hampshire, are contained in Appendix A and details of the Level of Service analysis are contained in Appendix B.

2. RESEARCH APPROACH

The basic study approach will be presented, followed by a discussion of the factors that may influence the performance of an acceleration lane and the identification of the factors that were considered in the study. Subsequently, additional issues relevant to the site selection will be discussed. Following, an outline of the tasks undertaken for the survey of existing practice as well as the operational and safety analysis will be presented. Finally, the approach to developing guidelines for implementation will be discussed.

2.1 Basic Study Approach

The study followed the approach proposed by Caltrans (2) i.e. a survey of existing practice and an operational and safety analysis of existing intersections with and without acceleration lanes.

The survey of existing practice regarding both the guidelines for implementation and the dimensions of acceleration lanes was carried out through a review of journal articles and other documents on existing practice; by requesting existing standards for acceleration lanes from all the other states and through correspondence and interviews with Caltrans personnel. The existing standards and practice are good evidence of what has been found acceptable, while the articles on the subject cast some light upon the underlying theory and any proposed practice which may not have been implemented through existing standards.

The ideal approach to the operational and safety analysis would be to measure the performance of an intersection without acceleration lanes; measure the performance of the same intersection with an acceleration lane and compare before and after performances. Several repetitions would be required for each type of intersection to make the results statistically reliable. A similar result could be obtained by comparing the performance of intersections with acceleration lanes to the performance of similar intersections without acceleration lanes.

A specific type of intersection would be defined by a set of values for the following characteristics:

For both the major highway and the cross road:

- - divided versus undivided
- number of lanes
- design speed
- gradient
- lane, shoulder and median width
- traffic volume and composition
- turning volumes
- horizontal curve radii

Adjacent intersections:

- proximity of adjacent intersections
- types of adjacent intersections

The ideal approach was, however, infeasible since the available

resources did not allow for such an extensive experiment. It was impossible to conduct before and after studies within the study period and it was also impossible to conduct studies for similar sites with and without acceleration lanes for all the possible different types of intersections. The appropriate approach to this project therefore became one of using the resources most effectively to attain the best results. This entailed carrying out data collection only once at a site representing one set of factors, limiting the number of characteristics investigated and also limiting the range of values of the characteristics.

2.2 Influencing Factors

Some of the factors that may influence the operational and safety performance of an acceleration lane are listed in section 2.1. Because of the limitation of resources and a lack of sites where the influence of some factors could be measured, only the most relevant factors and intersection types were considered.

Based on discussions with Caltrans, undivided four-lane highways were excluded from the study. The design speeds of the highways were not explicitly taken into account, due to limited resources. This could have been accomplished by comparing the performance of acceleration lanes on highways with different design speeds. The design speeds were, however, considered in discussions of the study results, where relevant.

acceleration lengths required by trucks and buses are considered to be unreasonably long for design purposes. The recommended taper lengths are shown in table 3.5. They are based on a time of three and a half seconds necessary to change lanes.

The 1965 Policy recommended lengths at a design speed of 40 mph are lower than Case I but higher than Case II of the 1954 value. At a design speed of 50 mph, the 1965 lengths are practically identical to the Case I value of 1954. For design speeds of 60 and 70 mph the 1965 recommended lengths are greater than both cases in the 1954 Policy. It is not apparent why the values changed since they were based on the same acceleration rates.

The position on providing acceleration lanes at stop controlled intersections is similar to that of the 1990 Policy. It is mentioned, however, that drivers will make little use of an acceleration lane although they utilize a short paved taper. When traffic volumes are relatively low, entering vehicles generally follow direct paths. Some traffic enters the highway without utilizing a large part of the acceleration lane, although greater usage is obtained with higher volumes. Acceleration lanes are therefore provided not only to permit increasing speed before entering the through traffic lanes but also to serve as maneuvering space so that a driver can take advantage of an opening in the adjacent stream of through traffic and move laterally into it. For this reason, as much of the speed-change facility as feasible should be adjacent to and flush with the through pavement. No barriers such as curbs between lane and shoulder should exist that would

make it difficult for a driver to continue on the shoulder if the opening in through traffic does not materialize.

When volumes are high most vehicles generally make full use of the acceleration lane. At the more important rural intersections where speeds and volumes make acceleration lanes appropriate, it is recommended to design an above-minimum radius and a corner island plan where the right turning entering traffic would be subject to yield sign control.

According to the 1965 Policy, a speed change lane of uniform width should not be less than 11 feet and preferably should be 12 feet wide.

The Caltrans Design Manuals

As stated in the introduction, the latest Caltrans Highway Design Manual (1) does not contain guidelines for the use of acceleration lanes. Earlier manuals did contain guidelines.

The 1952 Manual (9) stated that "Except as stated below, speed change lanes shall be provided for any individual turning movement when its design hourly volume is 25 or more vehicles."

The exception noted for acceleration lanes was for median acceleration lanes at signalized intersections. The recommended lengths are presented in Table 3.5. It is also stated that "When the design hourly volume of through traffic is more than 700 or more vehicles per lane,

TABLE 3.5: Caltrans Acceleration Lane Lengths (1952)

		Highway Design Speed [V] MPH		
		40	50	60
Average Speed of Travel [0.7V]		28	35	42
Acceleration		Taper - Feet		
		180	240	270
Turning Speed (mph)	Minimum Curve Radius (ft)	Acceleration Lane - Feet —Including Taper—		
20	100	180	410	750
30	200	180	240	510
40	400	180	240	270
50	600	—	240	270

acceleration lane lengths shall be increased 300 ft. above the values given.." A width of 12 ft. is recommended.

An amendment was made in 1957 to the recommended lengths. The amended lengths are shown in Table 3.6. These lengths are comparable to those used in later AASHTO Policies. If grades were steeper than four percent, lengths were to be increased by 300 ft.

The 1959 Manual contained the same narrative as the 1952 Manual regarding when an acceleration lane should be used and provided detailed guidelines for the taper design, but did not contain recommendations for the length of the acceleration lane.

Other Publications

According to Jouzy and Michael (10), acceleration lane traffic had little effect on the speed of through traffic at interchanges with acceleration lanes. "At only 8 of the 28 acceleration and deceleration lane locations studied was the difference between the 85th percentile speed of the through lane traffic within the area of conflict and that beyond the area of conflict statistically significant. At some areas where the speed effect was significant, other factors, such as a narrow median or a horizontal curve, probably contributed to the changes in speed". They also reported that a higher percentage of drivers utilized more length of the acceleration lane when "the acceleration lane met the through lane on a right curve, and less length of the acceleration lane when it met on a left curve, than under the condition where the

**TABLE 3.6: 1952 Caltrans Highway Design Manual Acceleration Lane Lengths
(As Amended, 1957)**

		Acceleration Lane, Including Taper (ft)			
Minimum Curve Radius (ft)		100	200	400	650
Turning Speed (m.p.h.)		20	30	40	50
Highway Design Speed (m.p.h.)	Taper Length (ft)				
70	300	900	750	500	*
60	270	750	510	*	*
50	240	410	*	*	*
40	180	*	*	*	*

* Acceleration Lane length shall include the taper length plus the distance required to move laterally from the width at the inlet nose to a width of 12 feet at the beginning of the taper.

acceleration lane met the through lane on a tangent".

Michael and Jouzy discovered that a large number of motorists apparently do not know how to use acceleration lanes properly. For the most efficient and safest operation of traffic they recommended that the driving public be better informed on the proper use of acceleration lanes.

Regarding taper length, Prisk (11) stated that an "... average of 3.5 to 4.5 sec. is used by most drivers to get their vehicles from a low running speed to the point of encroaching on the left lane". It is important to note that these vehicles were operating at a "low" speed and not accelerating from a stop condition. Prisk's paper was written in 1941 and the acceleration characteristics of today's vehicles could be quite different than the vehicles of that era.

A study performed by Sawhill and Neuzil (12) stated that two-way median left-turn lanes can serve as acceleration lanes for vehicles turning left onto arterial streets from minor streets and abutting properties. They also found that the two way left turn lanes facilitated the movement of the through traffic and provide a high degree of access service, yet their use did not result in an increase in traffic accidents.

Baker (13) used a similar approach to that followed in the AASHTO policies to arrive at minimum lengths for acceleration lanes which reflect vehicular speed and performance data of 1970 conditions. These lengths are shown in

Table 3.7 for relatively flat grades. Recommended length adjustments where grades equal or exceed three percent are shown in Table 3.8.

Reilly, Pfeffer, Michaels, Polus and Schoen (14) performed a study to determine speed-change lanes for freeways. They developed a procedure for determining the length of acceleration lanes which is based on the speed at an intermediate point on the on-ramp. The procedure resulted in lengths significantly greater than the AASHTO values. For illustrative purposes, one of their example designs is presented in Figure 3.1.

Median acceleration lanes are briefly discussed in an Institute of Transportation Engineers informational Report (15). References are made to studies by Van Winkle (16) and Blair (17). According to the report, little information exist on median acceleration lanes, but it mentions that there is some evidence that the lanes improve traffic flow and reduce accidents.

3.2 Survey of Practice in Other States

The objective of this survey was to review existing guidelines used by other states and other jurisdictions for implementing acceleration lanes for right-and left-turning traffic onto four-lane or two-lane rural high-speed highways at four-leg stop-controlled intersections.

A request for information was sent to the transportation official identified as the State Representative to the Transportation Research

TABLE 3.7: Lengths of Speed Change Lanes, R.F. Baker, 1975

Design Speed of Highway		Design Speed of Turning Roadway																	
		Stop		15 mph kph	24 mph kph	20 mph kph	32 mph kph	25 mph kph	40 mph kph	30 mph kph	48 mph kph	35 mph kph	56 mph kph	40 mph kph	64 mph kph	45 mph kph	72 mph kph	50 mph kph	80 mph kph
		Feet	Meters																
Minimum Deceleration Lane Lengths (Including Taper) ^b																			
40	64	325	99.1	300	91.4	275	83.8	250	76.2	200	61.0	—	—	—	—	—	—	—	—
50	80	425	129.5	400	121.9	375	114.3	350	106.7	325	99.1	275	83.8	—	—	—	—	—	—
60	97	500	152.4	500	152.4	475	144.8	450	137.2	425	129.5	400	121.9	325	99.1	300	91.4	—	—
70	113	600	182.9	575	175.3	550	167.6	550	167.6	525	160.0	500	152.4	425	129.5	400	121.9	350	106.7
80	129	700	213.4	675	205.7	650	198.1	650	198.1	600	182.9	575	175.3	525	160.0	475	144.8	450	137.2
Recommended Deceleration Lane Lengths (Including Taper) ^b																			
40	64	425	129.5	400	121.9	350	106.7	325	99.1	250	76.2	—	—	—	—	—	—	—	—
50	80	525	160.0	500	152.4	450	137.2	425	129.5	375	114.3	350	106.7	—	—	—	—	—	—
60	97	625	190.5	600	182.9	550	167.6	550	167.6	500	152.4	450	137.2	425	129.5	400	121.9	—	—
70	113	725	221.0	700	213.4	650	198.1	650	198.1	625	190.5	575	175.3	525	160.0	475	144.8	450	137.2
80	129	850	259.1	800	243.8	775	236.2	750	228.6	725	221.0	700	213.4	625	190.5	575	175.3	550	167.6
Minimum Acceleration Lane Lengths (Including Taper) ^b																			
40	64	—	—	325	99.1	250	76.2	225	68.6	—	—	—	—	—	—	—	—	—	—
50	80	—	—	700	213.4	625	190.5	600	182.9	500	152.4	400	121.9	—	—	—	—	—	—
60	97	—	—	1125	342.9	1075	327.7	1000	304.8	900	274.3	800	243.8	600	182.9	400	121.9	—	—
70	113	—	—	1550	472.4	1500	457.2	1400	426.7	1325	403.9	1225	373.4	1000	304.8	825	251.5	575	175.3
80	129	—	—	1975	602.0	1900	579.1	1825	556.3	1750	533.4	1650	502.9	1450	442.0	1250	381.0	1000	304.8
Recommended Acceleration Lane Lengths (Including Taper) ^b																			
40	64	—	—	425	129.5	325	99.1	300	91.4	—	—	—	—	—	—	—	—	—	—
50	80	—	—	900	274.3	800	243.8	775	236.2	650	198.1	525	160.0	—	—	—	—	—	—
60	97	—	—	1400	426.7	1300	396.2	1250	381.0	1125	342.9	1000	304.8	775	236.2	525	160.0	—	—
70	113	—	—	1875	571.5	1775	541.0	1725	525.8	1650	502.9	1500	457.2	1250	381.0	1000	304.8	750	228.6
80	129	—	—	2375	723.9	2275	693.4	2200	670.6	2100	640.1	1975	602.0	1750	533.4	1500	457.2	1200	365.8

^aLengths shown are for relatively flat grades. For 3% or greater grades, adjust as shown in Table 3.6.

^bFor design speed in mph, lengths shown in feet; in kph, lengths shown in meters.

Source: Modified Information for Minimum Length from AASHTO Bluebook, p. 351 (Ref. 5). Recommended lengths are author's.

TABLE 3.8: Speed Change Lane Length Adjustments for Grade, R.F. Baker, 1975

Highway Design Speed		Grade, %	Multiplier Factor For Turning Roadway Design Speed of:							
mph	kph		20 mph	32 kph	30 mph	48 kph	40 mph	64 kph	50 mph	80 kph
40	64	+3 to < +5	1.30	1.30	1.35	1.35	—	—	—	—
		Over +5	1.50	1.50	1.55	1.55	—	—	—	—
		-3 to < -5	0.70	0.70	0.70	0.70	—	—	—	—
		Over -5	0.60	0.60	0.60	0.60	—	—	—	—
50	80	+3 to < +5	1.35	1.35	1.40	1.40	1.45	1.45	—	—
		Over +5	1.55	1.55	1.70	1.70	1.90	1.90	—	—
		-3 to < -5	0.65	0.65	0.65	0.65	0.65	0.65	—	—
		Over -5	0.55	0.55	0.55	0.55	0.55	0.55	—	—
60	97	+3 to < +5	1.40	1.40	1.50	1.50	1.50	1.55	1.60	1.60
		Over +5	1.70	1.70	1.90	1.90	2.20	2.20	2.50	2.50
		-3 to < -5	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
		Over -5	0.50	0.50	0.50	0.50	0.60	0.50	0.50	0.50
70	113	+3 to < +5	1.50	1.50	1.60	1.60	1.70	1.70	1.80	1.80
		Over +5	2.00	2.00	2.20	2.20	2.60	2.60	3.00	3.00
		-3 to < -5	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
		Over -5	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
80	129	+3 to < +5	1.60	1.60	1.70	1.70	1.85	1.85	2.00	2.00
		Over +5	2.20	2.20	2.45	2.45	2.95	2.95	3.50	3.50
		-3 to < -5	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55
		Over -5	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
All	All	+3 to < +5 Over +5 -3 to < -5 Over -5	Deceleration Lanes							
			0.90	←						→ 0.90
			0.80	←						→ 0.80
			1.20	←						→ 1.20
			1.35	←						→ 1.35

Source: Data partially derived from AASHO Bluebook, p. 352. (Ref. 5)

FIGURE 3.1: Example Acceleration Lane Length Requirements, NCHRP 3-35

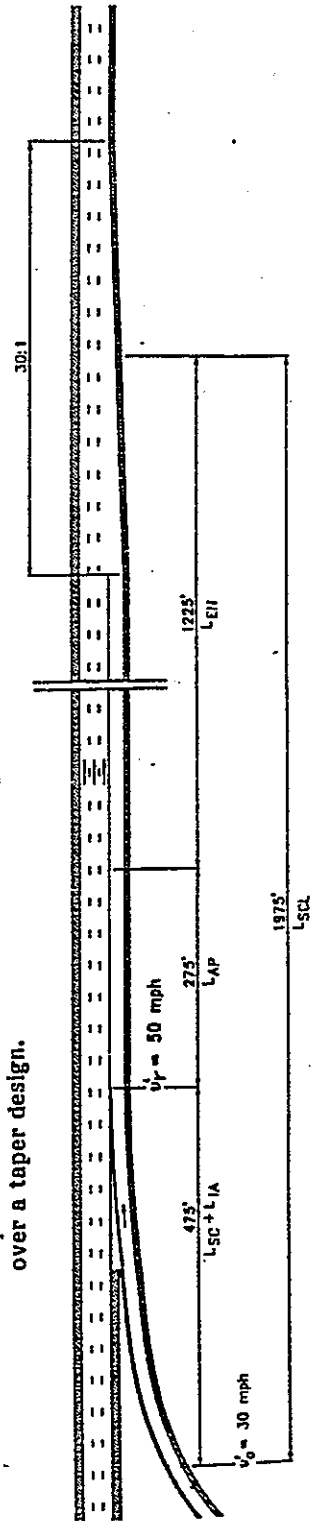
Length Requirements

Since the ramp speed at the begin GSA, v'_r , is unknown, the designer would try several speeds in order to obtain the shortest acceleration lane length. Therefore, for $v'_r = 60, 50, 40$, and 30 mph;

	$v'_r = 60$ mph	$v'_r = 50$ mph	$v'_r = 40$ mph	$v'_r = 30$ mph
From Table 1; $L_{SC} + L_{IA} =$	775 ft	475 ft	250 ft	50 ft
From Table 3; $L_{AP} =$	500 ft	275 ft	175 ft	100 ft
From Table 4; $L_{EN} =$	1,150 ft	1,225 ft	2,050 ft	2,325 ft
$L_{SCL} =$	2,425 ft	1,975 ft	2,475 ft	2,475 ft

Proposed Design

For this design, a $v'_r = 50$ mph would be selected, resulting in $L_{SCL} = 1,975$ ft. In order to meet the design constraints for placement of the acceleration lane elements, the following parallel design could be suggested. Based on the length requirement of the acceleration lane, a parallel design is chosen over a taper design.



Board in each of the states for which standards were not available and to the Transportation Research Board Representatives for Puerto Rico and Washington D.C.

Although this research is not directed at acceleration lanes at interchanges, it was felt that standards for these could also be helpful. For this reason, the request for information was made for standards for acceleration lanes at intersections as well as acceleration lanes at interchanges.

Responses

The responses are summarized in Table 3.9. Several of the thirty-four respondents, either in their letter of correspondence or in the literature they sent, made reference to one or more of the 1990 and 1984 editions of AASHTO's *A Policy on Geometric Design of Highways and Streets* (4) and AASHO's 1965 *A Policy on Geometric Design of Rural Highways* (8).

Twenty-two specifically included some type of standard for acceleration lanes at interchanges and six have specific guidelines for acceleration lanes at intersections.

The states that have no standards for the use of acceleration lanes at intersections on rural highways are: Arizona, Iowa (for two-lane roads), Maine, Massachusetts, Michigan, Minnesota, New York, Ohio and Pennsylvania.

TABLE 3.9: Response to Request for Standards

Name of States	State Responded	Included Standards for Accel. Lanes @ Intersections	Included Standards for Accel. Lanes @ Interchanges	Other Published Standards Referenced	Requested Final Copy of this Report
Alabama					
Alaska	Y	N	Y	none	Y
Arizona	Y	N	N	none	
Arkansas	Y	N	Y	none	
Colorado	Y	Y	Y	none	
Connecticut					
Delaware					
District of Columbia					
Florida	Y	Y	Y	AASHTO	
Georgia					
Hawaii					
Idaho	Y	N	Y	AASHTO Green Book, '90	
Illinois	Y	N	N	AASHTO Green Book, '84	
Indiana	Y	N	Driveway Stds	AASHTO Green Book, '84	
Iowa	Y	N	Y	AASHTO Green Book, '90	
Kansas	Y	Rt. Turn Lanes	Y	AASHTO	Y
Kentucky	Y	N	N	AASHTO Green Book	
Louisiana					
Maine	Y	N	Y	none	
Maryland	Y	N	N	AASHTO	Y
Massachusetts	Y	N	N	none	
Michigan	Y	N	Y	none	
Minnesota	Y	N	Y	none	
Mississippi					
Missouri	Y	N	Y	AASHTO Green Book	
Montana	Y	N	Y	AASHTO Green Book, '90	
Nebraska	Y	N	N	none	
Nevada					
New Hampshire	Y	N	Y	none	
New Jersey					
New Mexico					
New York	Y	N	Y	65 AASHO Policy on Geometric Design of Rural Highways	
North Carolina	Y	N	Y	none	
North Dakota					
Ohio	Y	N	Y	none	
Oklahoma					
Oregon	Y	N	Y	AASHTO	Y
Pennsylvania	Y	N	Y	none	
Puerto Rico					
Rhode Island	Y	N	N	AASHTO	
South Carolina	Y	N	N	AASHTO Green Book	Y
South Dakota					
Tennessee	Y	N	N	AASHTO Green Book	
Texas	Y	N	N	none	
Utah					
Vermont					
Virginia	Y	N	Y	AASHTO Green Book, '84	
Washington	Y	Rt. Turn Lanes	Y	AASHTO Green Book; HCM	
West Virginia	Y	N	Y	AASHTO	
Wisconsin	Y	Rt. Turn Lanes	Y	AASHTO Green Book	
Wyoming	Y	N	N	AASHTO Green Book, '90	Y

Massachusetts' documentation describes the reason for not having standards for these acceleration lanes:

"The Commonwealth of Massachusetts does not use acceleration lanes for right and left turning traffic on two and four lane rural highways at four-leg stop-controlled intersections. These acceleration lanes would serve right turning traffic fairly well; however, presently right turning traffic picks gaps in the oncoming traffic which allows them to accelerate without impacting the traffic flow.

Left turning traffic would have to cross traffic approaching from both the right and left directions to enter the acceleration lane. In most cases now, the left turning traffic crosses the left approaching traffic, while picking a suitable gap for accelerating into the right approaching flow.

Drivers are usually able to merge into traffic in this manner since these are rural roads and as such they have the necessary gaps in traffic flow to allow this movement. Where conflicts are insurmountable, signal control is generally preferred."

Texas verbally responded that the shoulder is sometimes utilized for acceleration purposes by right turning traffic.

The states that mention some use of acceleration lanes but do not

provide any specific reference to standards of acceleration lanes at intersections on rural highways are: Arkansas, Illinois, Iowa, Kansas, Maryland, Nebraska, North Carolina, Rhode Island and Virginia. Arkansas and Virginia do not typically provide acceleration lanes. Illinois does not utilize acceleration lanes at stop-controlled intersections. Iowa does not endorse the use of acceleration lanes but allows them on four-lane roads.

Kansas will not provide an acceleration lane at an intersection unless a special situation permits a free right turning movement. In such a case an acceleration lane could be considered appropriate. Maryland will not normally use an acceleration lane at an intersection unless there is a channelized right turn. In the event of providing a channelized right turn, an acceleration lane is generally provided.

Nebraska will not generally build an acceleration lane at a stop-controlled intersection except when the acceleration lane becomes part of an auxiliary lane.

The reasoning for not normally using acceleration lanes at intersections is illustrated by the comments received from North Carolina:

"It is felt that motorists require excessively long acceleration lanes at stop-controlled intersections. Moreover, as development occurs near the intersection, such acceleration lanes create inadequate weaving areas."

Rhode Island does not use acceleration lanes at stop-controlled intersections but only on high speed facilities (greater than 35 mph).

The states that follow AASHTO guidelines are: Idaho, Indiana, Kentucky, Missouri, Montana, Oregon, South Carolina, Tennessee, West Virginia, and Wyoming. Wyoming provided the following response:

"Wyoming Department of Transportation is currently using the 1990 Policy on Geometric Design of Highways and Streets (Green Book). We use table X-4 [See attachment A] Minimum Acceleration Length for entrance with flat grade of 2 percent or less page 986 on four lane or two lane rural highways at stop controlled intersections.

The state of Wyoming has low traffic counts on most of the rural highways and does not have their own standards for acceleration lanes on rural highways. The drivers in Wyoming usually wait for a gap to accelerate at right and left turns on rural highways. The district traffic engineers have recommended acceleration lanes at intersections where they deemed necessary because of safety."

The six states that have standards for acceleration lanes at intersections are: Alaska, Colorado, Florida, New Hampshire, Washington and Wisconsin. Alaska provided warrants for acceleration lanes in urban areas only. A summary of the specific guidelines for acceleration lanes at rural intersections which were provided, follow:

Colorado

The State Highway Access Code for the state was provided. This code gives warrants for the use of acceleration lanes for right turning vehicles at intersections. Their necessity is determined by the use of graphs of the average peak hour volume of vehicles turning onto the through road versus the volume of traffic per single lane of the through road (see Figure 3.2).

Guidelines for lane length, taper design and grade adjustment are also given. The lengths correspond to those recommended in the 1990 AASHTO Policy except that the posted speed is substituted for the design speed. The recommended taper ratios and grade adjustments are provided in Table 3.10.

Detailed guidelines for acceleration lanes for left turning traffic were not provided. If the warrants provided in Figure 3.2 are met and the posted speed is above 40 mph, then the acceleration lanes may be provided.

The recommended width of an acceleration lane is 12 ft. and should not be less than a minimum of 10 ft.

Florida

Extracts from the Florida Department of Transportation road design standards were received. The literature makes reference to AASHTO's A

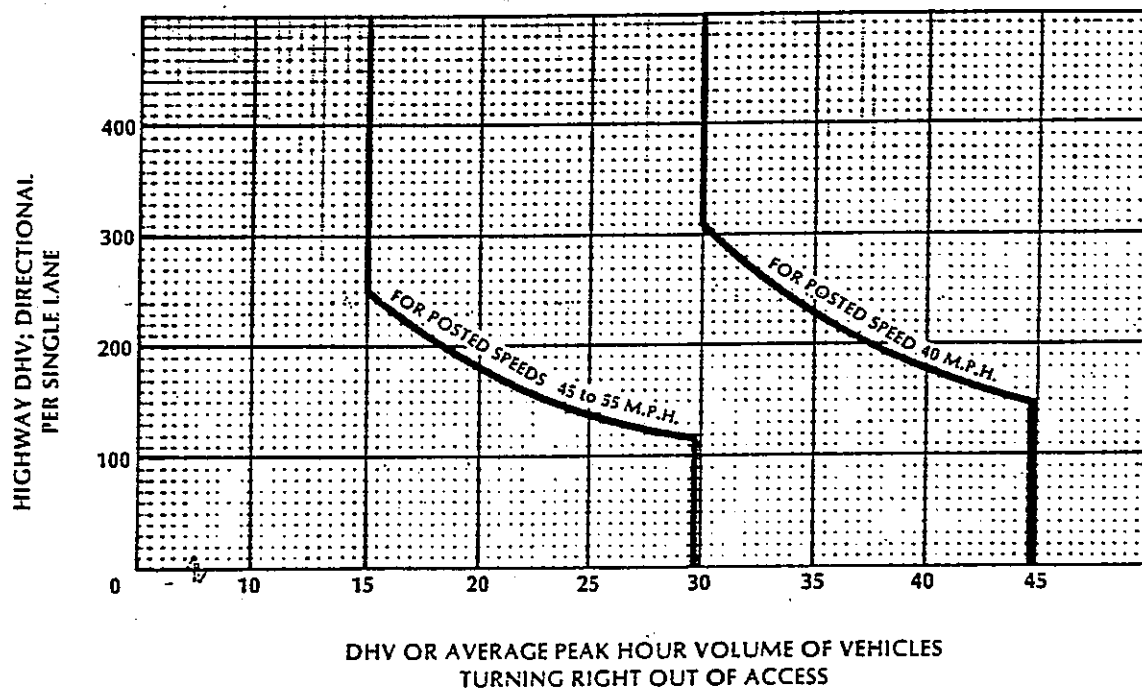
FIGURE 3.2: Colorado Acceleration Lane Guidelines

TABLE 3.10: Colorado Acceleration Lane Lengths

Posted Speed (mph)	Acceleration Lane Length Following a Stop or Turn (ft)		Ratio for Straight Taper	Grade Adjustment Factors				Redirect Taper Ratios (Not included in Acceleration Lane Lengths)
	Stop Condition	15 MPH Turn		3 to 4.9% Upgrade	3 to 4.9% Downgrade	5 to 7% Upgrade	5 to 7% Downgrade	
25	100	150	7.5:1	1.3	0.7	1.5	0.6	20:01
30	190	185	10:01	1.3	0.7	1.5	0.6	20:01
35	270	235	12.5:1	1.3	0.7	1.5	0.6	25:1
40	380	295	15:01	1.3	0.7	1.5	0.6	30:1
45	550	350	15:01	1.3	0.7	1.5	0.6	40:1
50	760	405	20:01	1.4	0.65	1.8	0.55	45:1
55	960	450	22.5:1	1.5	0.65	2	0.55	50:1
60				1.5	0.6	2.3	0.5	50:1

acceleration lane lengths shall be increased 300 ft. above the values given.." A width of 12 ft. is recommended.

An amendment was made in 1957 to the recommended lengths. The amended lengths are shown in Table 3.6. These lengths are comparable to those used in later AASHTO Policies. If grades were steeper than four percent, lengths were to be increased by 300 ft.

The 1959 Manual contained the same narrative as the 1952 Manual regarding when an acceleration lane should be used and provided detailed guidelines for the taper design, but did not contain recommendations for the length of the acceleration lane.

Other Publications

According to Jouzy and Michael (10), acceleration lane traffic had little effect on the speed of through traffic at interchanges with acceleration lanes. "At only 8 of the 28 acceleration and deceleration lane locations studied was the difference between the 85th percentile speed of the through lane traffic within the area of conflict and that beyond the area of conflict statistically significant. At some areas where the speed effect was significant, other factors, such as a narrow median or a horizontal curve, probably contributed to the changes in speed". They also reported that a higher percentage of drivers utilized more length of the acceleration lane when "the acceleration lane met the through lane on a right curve, and less length of the acceleration lane when it met on a left curve, than under the condition where the

**TABLE 3.6: 1952 Caltrans Highway Design Manual Acceleration Lane Lengths
(As Amended, 1957)**

		Acceleration Lane, Including Taper (ft)			
Minimum Curve Radius (ft)		100	200	400	650
Turning Speed (m.p.h.)		20	30	40	50
Highway Design Speed (m.p.h.)	Taper Length (ft)				
70	300	900	750	500	*
60	270	750	510	*	*
50	240	410	*	*	*
40	180	*	*	*	*

* Acceleration Lane length shall include the taper length plus the distance required to move laterally from the width at the inlet nose to a width of 12 feet at the beginning of the taper.

acceleration lane met the through lane on a tangent".

Michael and Jouzy discovered that a large number of motorists apparently do not know how to use acceleration lanes properly. For the most efficient and safest operation of traffic they recommended that the driving public be better informed on the proper use of acceleration lanes.

Regarding taper length, Prisk (11) stated that an "... average of 3.5 to 4.5 sec. is used by most drivers to get their vehicles from a low running speed to the point of encroaching on the left lane". It is important to note that these vehicles were operating at a "low" speed and not accelerating from a stop condition. Prisk's paper was written in 1941 and the acceleration characteristics of today's vehicles could be quite different than the vehicles of that era.

A study performed by Sawhill and Neuzil (12) stated that two-way median left-turn lanes can serve as acceleration lanes for vehicles turning left onto arterial streets from minor streets and abutting properties. They also found that the two way left turn lanes facilitated the movement of the through traffic and provide a high degree of access service, yet their use did not result in an increase in traffic accidents.

Baker (13) used a similar approach to that followed in the AASHTO policies to arrive at minimum lengths for acceleration lanes which reflect vehicular speed and performance data of 1970 conditions. These lengths are shown in

Table 3.7 for relatively flat grades. Recommended length adjustments where grades equal or exceed three percent are shown in Table 3.8.

Reilly, Pfeffer, Michaels, Polus and Schoen (14) performed a study to determine speed-change lanes for freeways. They developed a procedure for determining the length of acceleration lanes which is based on the speed at an intermediate point on the on-ramp. The procedure resulted in lengths significantly greater than the AASHTO values. For illustrative purposes, one of their example designs is presented in Figure 3.1.

Median acceleration lanes are briefly discussed in an Institute of Transportation Engineers informational Report (15). References are made to studies by Van Winkle (16) and Blair (17). According to the report, little information exist on median acceleration lanes, but it mentions that there is some evidence that the lanes improve traffic flow and reduce accidents.

3.2 Survey of Practice in Other States

The objective of this survey was to review existing guidelines used by other states and other jurisdictions for implementing acceleration lanes for right-and left-turning traffic onto four-lane or two-lane rural high-speed highways at four-leg stop-controlled intersections.

A request for information was sent to the transportation official identified as the State Representative to the Transportation Research

TABLE 3.7: Lengths of Speed Change Lanes, R.F. Baker, 1975

Design Speed of Highway		Design Speed of Turning Roadway																	
		Stop		15	24	20	32	25	40	30	48	35	56	40	64	45	72	50	80
		Feet	Meters	mph	kph	mph	kph	mph	kph	mph	kph	mph	kph	mph	kph	mph	kph	mph	kph
Minimum Deceleration Lane Lengths (Including Taper) ^b																			
40	64	325	99.1	300	91.4	275	83.8	250	76.2	200	61.0	—	—	—	—	—	—	—	—
50	80	425	129.5	400	121.9	375	114.3	350	106.7	325	99.1	275	83.8	—	—	—	—	—	—
60	97	500	152.4	500	152.4	475	144.8	450	137.2	425	129.5	400	121.9	325	99.1	300	91.4	—	—
70	113	600	182.9	575	175.3	550	167.6	550	167.6	525	160.0	500	152.4	425	129.5	400	121.9	350	106.7
80	129	700	213.4	675	205.7	650	198.1	650	198.1	600	182.9	575	175.3	525	160.0	475	144.8	450	137.2
Recommended Deceleration Lane Lengths (Including Taper) ^b																			
40	64	425	129.5	400	121.9	350	106.7	325	99.1	250	76.2	—	—	—	—	—	—	—	—
50	80	525	160.0	500	152.4	450	137.2	425	129.5	375	114.3	350	106.7	—	—	—	—	—	—
60	97	625	190.5	600	182.9	550	167.6	550	167.6	500	152.4	450	137.2	425	129.5	400	121.9	—	—
70	113	725	221.0	700	213.4	650	198.1	650	198.1	625	190.5	575	175.3	525	160.0	475	144.8	450	137.2
80	129	850	259.1	800	243.8	775	236.2	750	228.6	725	221.0	700	213.4	625	190.5	575	175.3	550	167.6
Minimum Acceleration Lane Lengths (Including Taper) ^b																			
40	64	—	—	325	99.1	250	76.2	225	68.6	—	—	—	—	—	—	—	—	—	—
50	80	—	—	700	213.4	625	190.5	600	182.9	500	152.4	400	121.9	—	—	—	—	—	—
60	97	—	—	1125	342.9	1075	327.7	1000	304.8	900	274.3	800	243.8	600	182.9	400	121.9	—	—
70	113	—	—	1550	472.4	1500	457.2	1400	426.7	1325	403.9	1225	373.4	1000	304.8	825	251.5	575	175.3
80	129	—	—	1975	602.0	1900	579.1	1825	556.3	1750	533.4	1650	502.9	1450	442.0	1250	381.0	1000	304.8
Recommended Acceleration Lane Lengths (Including Taper) ^b																			
40	64	—	—	425	129.5	325	99.1	300	91.4	—	—	—	—	—	—	—	—	—	—
50	80	—	—	900	274.3	800	243.8	775	236.2	650	198.1	525	160.0	—	—	—	—	—	—
60	97	—	—	1400	426.7	1300	396.2	1250	381.0	1125	342.9	1000	304.8	775	236.2	525	160.0	—	—
70	113	—	—	1875	571.5	1775	541.0	1725	525.8	1650	502.9	1500	457.2	1250	381.0	1000	304.8	750	228.6
80	129	—	—	2375	723.9	2275	693.4	2200	670.6	2100	640.1	1975	602.0	1750	533.4	1500	457.2	1200	365.8

^aLengths shown are for relatively flat grades. For 2% or greater grades, adjust as shown in Table 9.6.

^bFor design speed in mph, lengths shown in feet; in kph, lengths shown in meters.

Source: Modified Information for Minimum Length from AASHTO Bluebook, p. 351 (Ref. 5). Recommended lengths are author's.

TABLE 3.8: Speed Change Lane Length Adjustments for Grade, R.F. Baker, 1975

Highway Design Speed		Grade, %	Multiplier Factor For Turning Roadway Design Speed of:								
mph	kph		20 mph	32 kph	30 mph	48 kph	40 mph	64 kph	50 mph	80 kph	
40	64	+3 to < +5	1.30	1.30	1.35	1.35	—	—	—	—	
		Over +5	1.50	1.50	1.55	1.55	—	—	—	—	
		-3 to < -5	0.70	0.70	0.70	0.70	—	—	—	—	
		Over -5	0.60	0.60	0.60	0.60	—	—	—	—	
50	80	+3 to < +5	1.35	1.35	1.40	1.40	1.45	1.45	—	—	
		Over +5	1.55	1.55	1.70	1.70	1.90	1.90	—	—	
		-3 to < -5	0.65	0.65	0.65	0.65	0.65	0.65	—	—	
		Over -5	0.55	0.55	0.55	0.55	0.55	0.55	—	—	
60	97	+3 to < +5	1.40	1.40	1.50	1.50	1.50	1.55	1.60	1.60	
		Over +5	1.70	1.70	1.90	1.90	2.20	2.20	2.50	2.50	
		-3 to < -5	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	
		Over -5	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	
70	113	+3 to < +5	1.50	1.50	1.60	1.60	1.70	1.70	1.80	1.80	
		Over +5	2.00	2.00	2.20	2.20	2.60	2.60	3.00	3.00	
		-3 to < -5	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	
		Over -5	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	
80	129	+3 to < +5	1.60	1.60	1.70	1.70	1.85	1.85	2.00	2.00	
		Over +5	2.20	2.20	2.45	2.45	2.95	2.95	3.50	3.50	
		-3 to < -5	0.55	0.55	0.55	0.55	0.55	0.55	0.55	0.55	
		Over -5	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	
All	All	+3 to < +5	0.90	Deceleration Lanes							
		Over +5	0.80								0.90
		-3 to < -5	1.20								0.80
		Over -5	1.35								1.20
											1.35

Source: Data partially derived from AASHO Bluebook, p. 352. (Ref. 5)

Source: Data partially derived from AASHO Bluebook, p. 352. (Ref. 5)

FIGURE 3.1: Example Acceleration Lane Length Requirements, NCHRP 3-35

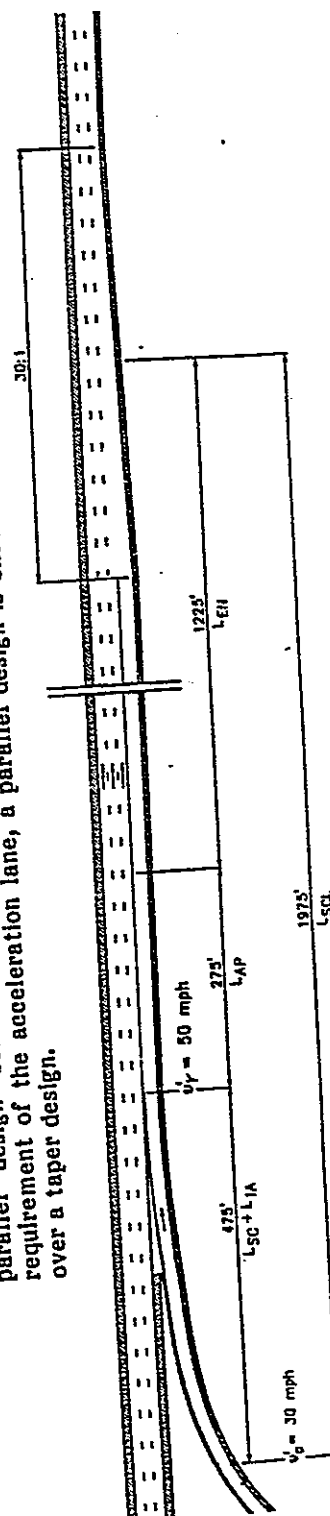
Length Requirements

Since the ramp speed at the begin GSA, v_r' , is unknown, the designer would try several speeds in order to obtain the shortest acceleration lane length. Therefore, for $v_r' = 60, 50, 40$, and 30 mph;

	$v_r' = 60$ mph	$v_r' = 50$ mph	$v_r' = 40$ mph	$v_r' = 30$ mph
From Table 1; $L_{SC} + L_{IA} =$	775 ft	475 ft	250 ft	50 ft
From Table 3; $L_{AP} =$	500 ft	275 ft	175 ft	100 ft
From Table 4; $L_{EN} =$	1,150 ft	1,225 ft	2,050 ft	2,325 ft
$L_{SCL} =$	2,425 ft	1,975 ft	2,475 ft	2,475 ft

Proposed Design

For this design, a $v_r' = 50$ mph would be selected, resulting in $L_{SCL} = 1,975$ ft. In order to meet the design constraints for placement of the acceleration lane elements, the following parallel design could be suggested. Based on the length requirement of the acceleration lane, a parallel design is chosen over a taper design.



Board in each of the states for which standards were not available and to the Transportation Research Board Representatives for Puerto Rico and Washington D.C.

Although this research is not directed at acceleration lanes at interchanges, it was felt that standards for these could also be helpful. For this reason, the request for information was made for standards for acceleration lanes at intersections as well as acceleration lanes at interchanges.

Responses

The responses are summarized in Table 3.9. Several of the thirty-four respondents, either in their letter of correspondence or in the literature they sent, made reference to one or more of the 1990 and 1984 editions of AASHTO's *A Policy on Geometric Design of Highways and Streets* (4) and AASHO's 1965 *A Policy on Geometric Design of Rural Highways* (8).

Twenty-two specifically included some type of standard for acceleration lanes at interchanges and six have specific guidelines for acceleration lanes at intersections.

The states that have no standards for the use of acceleration lanes at intersections on rural highways are: Arizona, Iowa (for two-lane roads), Maine, Massachusetts, Michigan, Minnesota, New York, Ohio and Pennsylvania.

TABLE 3.9: Response to Request for Standards

Name of States	State Responded	Included Standards for Accel. Lanes @ Intersections	Included Standards for Accel. Lanes @ Interchanges	Other Published Standards Referenced	Requested Final Copy of this Report
Alabama					
Alaska	Y				
Arizona	Y	N	Y	none	
Arkansas	Y	N	N	none	
Colorado	Y	N	Y	none	Y
Connecticut		Y	Y	none	
Delaware					
District of Columbia					
Florida	Y				
Georgia		Y	Y	AASHTO	
Hawaii					
Idaho	Y	N			
Illinois	Y	N	Y	AASHTO Green Book, '90	
Indiana	Y	N	N	AASHTO Green Book, '84	
Iowa	Y	N	Driveway Stds	AASHTO Green Book, '84	
Kansas	Y		Y	AASHTO Green Book, '90	
Kentucky	Y	Rt. Turn Lanes	Y	AASHTO	
Louisiana		N	N	AASHTO Green Book	Y
Maine	Y				
Maryland	Y	N	Y	none	
Massachusetts	Y	N	N	AASHTO	
Michigan	Y	N	N	none	Y
Minnesota	Y	N	Y	none	
Mississippi		N	Y	none	
Missouri	Y	N			
Montana	Y	N	Y	AASHTO Green Book	
Nebraska	Y	N	Y	AASHTO Green Book, '90	
Nevada		N	N	none	
New Hampshire	Y				
New Jersey		N	Y	none	
New Mexico					
New York	Y	N	Y		
North Carolina	Y			65 AASHO Policy on Geometric Design of Rural Highways	
North Dakota		N	Y	none	
Ohio	Y	N			
Oklahoma			Y	none	
Oregon	Y	N			
Pennsylvania	Y	N	Y	AASHTO	
Puerto Rico		N	Y	none	Y
Rhode Island	Y	N			
South Carolina	Y	N	N	AASHTO	
South Dakota			N	AASHTO Green Book	
Tennessee	Y	N			Y
Texas	Y	N	N	AASHTO Green Book	
Utah		N	N	none	
Vermont					
Virginia	Y	N			
Washington	Y	Rt. Turn Lanes	Y	AASHTO Green Book, '84	
West Virginia	Y	N	Y	AASHTO Green Book, HCM	
Wisconsin	Y	Rt. Turn Lanes	Y	AASHTO	
Wyoming	Y	N	N	AASHTO Green Book	
				AASHTO Green Book, '90	Y

Massachusetts' documentation describes the reason for not having standards for these acceleration lanes:

"The Commonwealth of Massachusetts does not use acceleration lanes for right and left turning traffic on two and four lane rural highways at four-leg stop-controlled intersections. These acceleration lanes would serve right turning traffic fairly well; however, presently right turning traffic picks gaps in the oncoming traffic which allows them to accelerate without impacting the traffic flow.

Left turning traffic would have to cross traffic approaching from both the right and left directions to enter the acceleration lane. In most cases now, the left turning traffic crosses the left approaching traffic, while picking a suitable gap for accelerating into the right approaching flow.

Drivers are usually able to merge into traffic in this manner since these are rural roads and as such they have the necessary gaps in traffic flow to allow this movement. Where conflicts are insurmountable, signal control is generally preferred."

Texas verbally responded that the shoulder is sometimes utilized for acceleration purposes by right turning traffic.

The states that mention some use of acceleration lanes but do not

provide any specific reference to standards of acceleration lanes at intersections on rural highways are: Arkansas, Illinois, Iowa, Kansas, Maryland, Nebraska, North Carolina, Rhode Island and Virginia. Arkansas and Virginia do not typically provide acceleration lanes. Illinois does not utilize acceleration lanes at stop-controlled intersections. Iowa does not endorse the use of acceleration lanes but allows them on four-lane roads.

Kansas will not provide an acceleration lane at an intersection unless a special situation permits a free right turning movement. In such a case an acceleration lane could be considered appropriate. Maryland will not normally use an acceleration lane at an intersection unless there is a channelized right turn. In the event of providing a channelized right turn, an acceleration lane is generally provided.

Nebraska will not generally build an acceleration lane at a stop-controlled intersection except when the acceleration lane becomes part of an auxiliary lane.

The reasoning for not normally using acceleration lanes at intersections is illustrated by the comments received from North Carolina:

"It is felt that motorists require excessively long acceleration lanes at stop-controlled intersections. Moreover, as development occurs near the intersection, such acceleration lanes create inadequate weaving areas."

Rhode Island does not use acceleration lanes at stop-controlled intersections but only on high speed facilities (greater than 35 mph).

The states that follow AASHTO guidelines are: Idaho, Indiana, Kentucky, Missouri, Montana, Oregon, South Carolina, Tennessee, West Virginia, and Wyoming. Wyoming provided the following response:

"Wyoming Department of Transportation is currently using the 1990 Policy on Geometric Design of Highways and Streets (Green Book). We use table X-4 [See attachment A] Minimum Acceleration Length for entrance with flat grade of 2 percent or less page 986 on four lane or two lane rural highways at stop controlled intersections.

The state of Wyoming has low traffic counts on most of the rural highways and does not have their own standards for acceleration lanes on rural highways. The drivers in Wyoming usually wait for a gap to accelerate at right and left turns on rural highways. The district traffic engineers have recommended acceleration lanes at intersections where they deemed necessary because of safety."

The six states that have standards for acceleration lanes at intersections are: Alaska, Colorado, Florida, New Hampshire, Washington and Wisconsin. Alaska provided warrants for acceleration lanes in urban areas only. A summary of the specific guidelines for acceleration lanes at rural intersections which were provided, follow:

Colorado

The State Highway Access Code for the state was provided. This code gives warrants for the use of acceleration lanes for right turning vehicles at intersections. Their necessity is determined by the use of graphs of the average peak hour volume of vehicles turning onto the through road versus the volume of traffic per single lane of the through road (see Figure 3.2).

Guidelines for lane length, taper design and grade adjustment are also given. The lengths correspond to those recommended in the 1990 AASHTO Policy except that the posted speed is substituted for the design speed. The recommended taper ratios and grade adjustments are provided in Table 3.10.

Detailed guidelines for acceleration lanes for left turning traffic were not provided. If the warrants provided in Figure 3.2 are met and the posted speed is above 40 mph, then the acceleration lanes may be provided.

The recommended width of an acceleration lane is 12 ft. and should not be less than a minimum of 10 ft.

Florida

Extracts from the Florida Department of Transportation road design standards were received. The literature makes reference to AASHTO's A

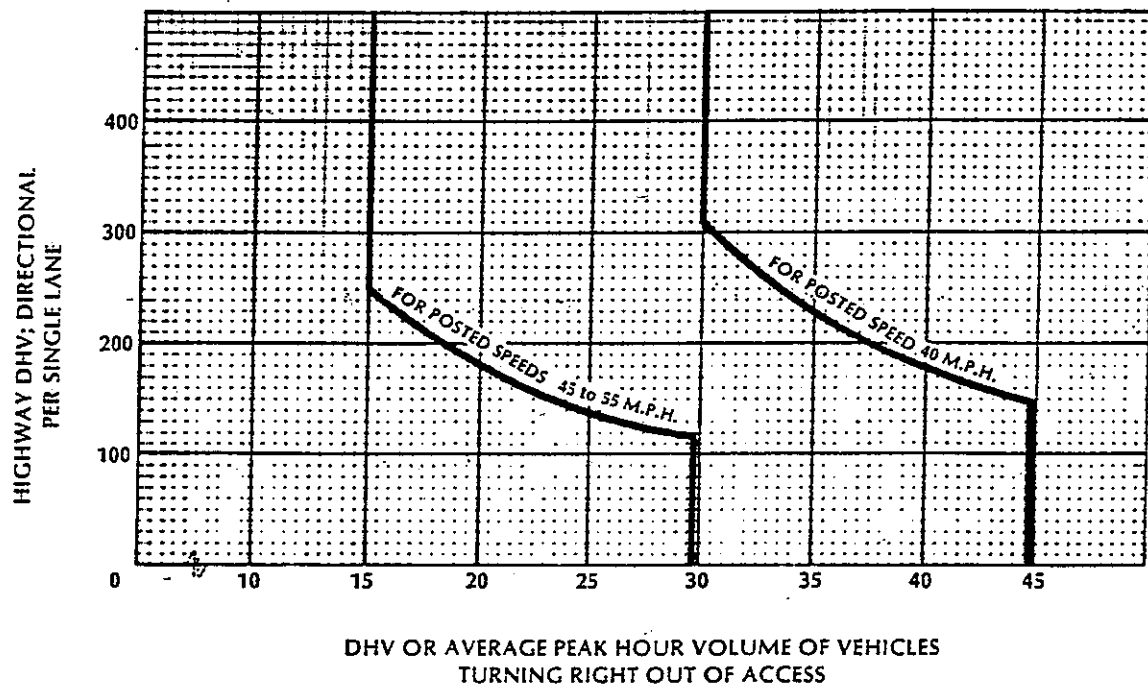
FIGURE 3.2: Colorado Acceleration Lane Guidelines

TABLE 3.10: Colorado Acceleration Lane Lengths

Posted Speed (mph)	Acceleration Lane Length Following a Stop or Turn (ft)		Ratio for Straight Taper	Grade Adjustment Factors				Redirect Taper Ratios (Not included in Acceleration Lane Lengths)
	Stop Condition	15 MPH Turn		3 to 4.9% Upgrade	3 to 4.9% Downgrade	5 to 7% Upgrade	5 to 7% Downgrade	
25	100	150	7.5:1	1.3	0.7	1.5	0.6	20:01
30	190	185	10:01	1.3	0.7	1.5	0.6	20:01
35	270	235	12.5:1	1.3	0.7	1.5	0.6	25:1
40	380	295	15:01	1.3	0.7	1.5	0.6	30:1
45	550	350	15:01	1.3	0.7	1.5	0.6	40:1
50	760	405	20:01	1.4	0.65	1.8	0.55	45:1
55	960	450	22.5:1	1.5	0.65	2	0.55	50:1
60				1.5	0.6	2.3	0.5	50:1

Policy on Geometric Design of Highways and Streets.

The literature recommends that "any street or highway with a large percentage of truck traffic should be considered for acceleration lane entrances."

Recommended design lengths and adjustments for grades for speed change lanes was included, which is identical to the lengths recommended in the 1965 AASHTO Policy. The values for taper length also corresponds to the 1965 AASHTO policy values. It is recommended that the merging taper be on a 50:1 transition but the length shall not be less than that set forth in the 1965 AASHTO Policy.

Lanes should be 12 ft. wide and not less than 10 ft. Streets and highways with significant truck traffic should have 12 feet wide traffic lanes.

New Hampshire

According to the documentation received, New Hampshire provides for acceleration in the following way:

"Our standard treatment at stop controlled rural intersections is to provide an additional 10' of travelway through the intersection (or stripe out existing 10' shoulder) and guide right turning traffic into travelway along a 10:1 taper (100'). In cases where the right turning volume is very heavy and this movement needs to

remain free flowing, a layout similar to Figure 1.D may be employed with the taper rate dependent on design speed and site rate dependent on design speed and site constraints."

The included Figure 1.D (see Appendix A) shows the preferred layout. Additional intersection layouts provided by New Hampshire are also provided in Appendix A.

Washington

Excerpts from the Washington State Design Manual were received. Guidelines for design are provided for right turn acceleration lanes for turning traffic with design speeds of less than 20 mph. The values for lengths, adjustments for grade, taper and width are similar to the 1990 AASHTO Policy for interchanges. For turning traffic design speeds in excess of 20 mph, the acceleration lane should be designed as a ramp.

Wisconsin

Standard drawings of "At-Grade Side Road Intersections" were provided. These drawings illustrated four types of T-intersections. Acceleration lanes are not shown, but dimensions for a taper for right turning traffic are provided.

When current traffic volumes exceed 2500 ADT on the through highway and 1000 ADT on the side road, a 100 foot taper length (converging over a width of 12 feet) that could serve as the acceleration taper for right

turning vehicles should be provided. When current traffic volumes on both the through highway and the side road exceed 100 ADT and the sum of both exceeds 1250 ADT, a taper of 100 feet (converging over a width of ten feet) should be provided.

At intersections not meeting warrants an acceleration taper of 10W feet (converging over a width of W feet) should be provided. "W" is equal to either the shoulder width or a minimum of 5 ft.

3.3 Survey of Practice in Caltrans Districts

All Caltrans districts were contacted for the purpose of obtaining guidelines used for the design of acceleration lanes. Information was obtained from most districts through personal interviews, telephonic conversations and when requested, through correspondence.

Summary of Results

None of the districts responding had specific guidelines for the use of acceleration lanes. Nevertheless, there appears to be some design practices that are noteworthy. These appear to be more developed for four-lane roads than for two-lane roads. The major issues and points of interest that emerged from the survey follow:

(a) Appropriate Use

It was ascertained that several districts have utilized acceleration

lanes at stop controlled intersections. Acceleration lanes existed in Districts 1,2,3,4,5,7,9 and 11.

It appeared that the districts considered the use of acceleration lanes on a case by case basis. Some instances of use, however, are noteworthy.

In District 9, most of the left turn acceleration lanes occur at "T" intersections. In District 5, short left turn acceleration lanes were created by striping the "bulb" area remaining from the widening of the road for the median turn lane on the main line, at the opposite side of the intersection, as an acceleration lane. The district observes the traffic turning left into these areas and if these areas are consistently used for acceleration, they may be striped as acceleration lanes.

District 4 uses the term "refuge" area instead of acceleration lane for the lower standard median acceleration lanes. The reason for this is that these short lanes do not provide adequate distance for acceleration but allow for the left turning vehicle to wait for an appropriate gap in the major road flow to permit them to enter the through lane.

Some Caltrans districts have reservations about providing acceleration lanes on two-lane rural roads. District 2 does not, as a matter of policy, actively create acceleration lanes for two-lane roads unless space is available. In Districts 4 and 5 it is believed that the use of acceleration lanes on two-lane roads does not decrease potential

conflicts, rather the locations of potential conflicts are simply moved.

(b) Length

Length is often governed by what is available. In District 1 the typical length ranges between 500 and 1000 ft. and could be longer if space and cost were not factors for both right turn and left turn acceleration lanes.

Districts 4 and 7 referred to the standard entrance ramp for freeways, without an auxiliary lane, in the California Highway Design Manual as having the desired lengths for acceleration lane design.

(c) Shoulder and Lane Width

The width of an acceleration lane should be the standard lane width of 12 feet or a minimum of 10 feet. In District 8, wide shoulders (8 ft.) are considered advantageous, since right turning vehicles often use the shoulder for acceleration purposes.

(d) Taper

District 3 uses as a rule of thumb the taper rate being the inverse of the speed in mph. For example, if the speed is 50 mph, then the taper rate is 1:50.

(e) Safety

A minimum safety index value of 200 is required to make any improvement on a highway classified as a safety project.

(e) Driver Behavior

In some districts it is felt that most drivers do not know how to use acceleration lanes. District 11 is trying to get away from the use of acceleration lanes because of the drivers' difficulty in using them. In the case of right turn acceleration lanes, it may be more difficult for drivers to look to the left and rear when merging, than looking to the left only in the absence of an acceleration lane.

Districts 1, 4, 5, 8, and 11 all stated that there are some operational difficulties with acceleration lanes. Acceleration lanes are felt not to remove conflicts but simply move them. Drivers on the acceleration lane and on the main line have difficulty with them.

3.4 Summary of Major Conclusions

A summary of major conclusions, based on the literature review, survey of practice in other states and Caltrans, follows:

1. The 1990 AASHTO Policy (4) does not have definitive guidelines on when to recommend the use of acceleration lanes and addresses primarily acceleration lanes for right turning vehicles. The same

lengths and taper lengths as those used for acceleration lanes at interchanges are recommended.

2. The lengths recommended for acceleration lanes in the 1990 AASHTO Policy are based on the same passenger car characteristics as used in the 1965 and 1954 policies, although new acceleration characteristics are presented in the 1990 Policy. According to the 1965 Policy, the acceleration lengths for trucks and buses are considered to be unreasonably long for design purposes.
3. The behavior of drivers in acceleration lanes is described in the 1965 AASHTO Policy. Drivers will make little use of an acceleration lane although they utilize a short paved taper. When traffic volumes are relatively low, entering vehicles generally follow direct paths. Some traffic enters the highway without utilizing a large part of the acceleration lane, although greater usage is obtained with higher volumes. "Acceleration lanes are therefore provided not only to permit increasing speed before entering the through traffic lanes but also to serve as maneuvering space so that a driver can take advantage of an opening in the adjacent stream of through traffic and move laterally into it.

When volumes are high most vehicles generally make full use of the acceleration lane." At the more important rural intersections where speeds and volumes make acceleration lanes appropriate, it is recommended to design an above-minimum radius and a corner

island plan where the right turning entering traffic would be subject to yield sign control.

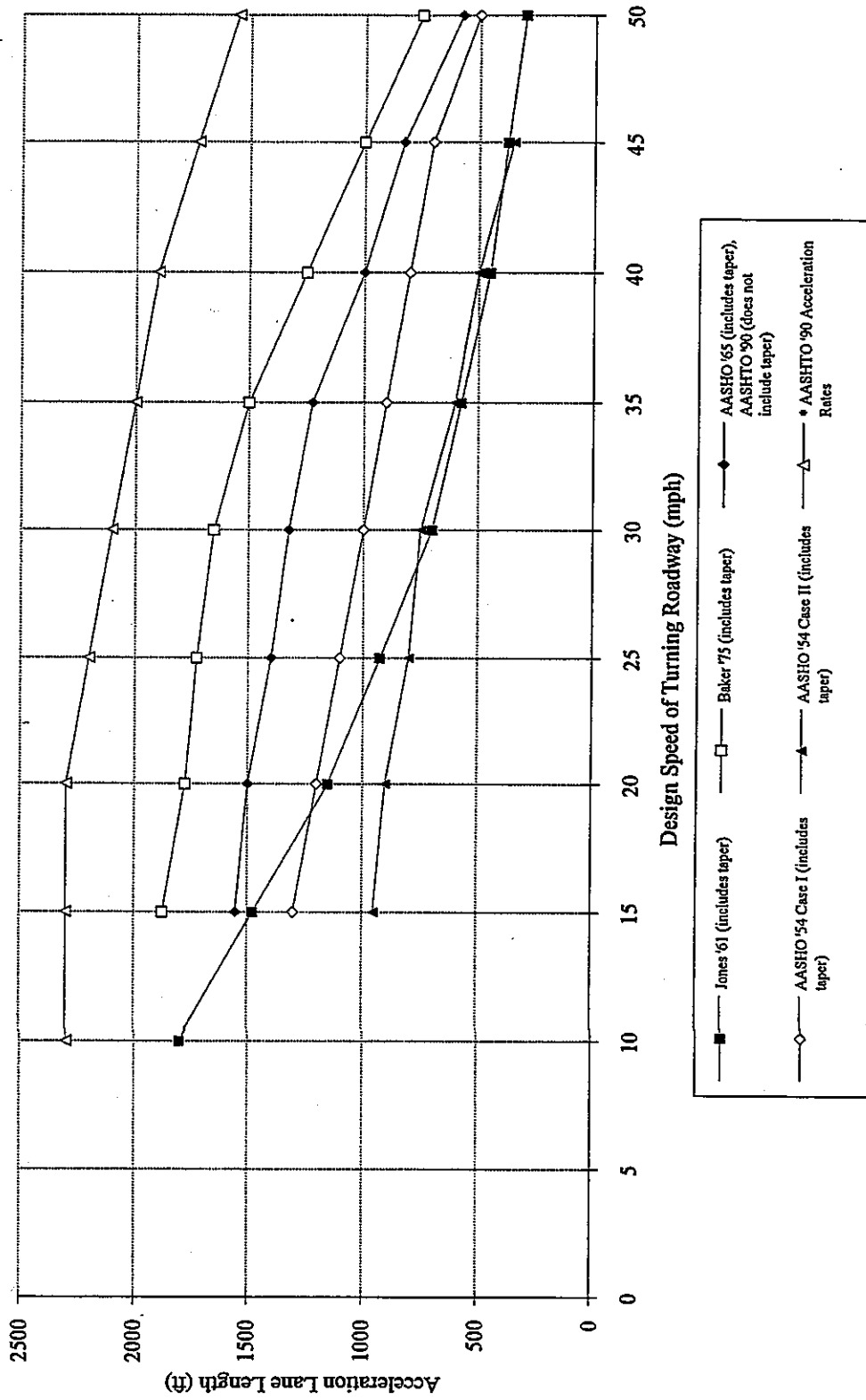
4. The Caltrans Highway Design Manual of 1990 (1) does not contain guidelines for acceleration lanes. The latest design manuals that contained comprehensive guidelines was the 1952 manual. When any individual turning movement had a design hourly volume of 25 or more vehicles, acceleration lanes had to be provided. The lengths of the acceleration lanes (as amended in 1957) were comparable to the AASHTO lengths. The recommended width was 12 ft.
5. Besides for the AASHTO policies, few other documents exist which discuss acceleration lanes. From these documents, the following are the major relevant conclusions that can be made:
 - ° Acceleration lanes do not affect the speeds of through traffic.
 - ° Motorists do not know how to use acceleration lanes effectively.
 - ° Two-way median left turn lanes can be used as acceleration lanes without increasing accidents.
6. According to the literature, the length of an acceleration lane (mostly discussed for freeway on-ramps) can be based on three factors in combination: the speed at which drivers merge with through traffic, the speed at which drivers enter the acceleration lanes and the layout of the acceleration lane. Adjustment of the

length should be made for grade changes. Values contained in the 1990 AASHTO Policy (4) are shown in Tables 3.1 and 3.2.

Figure 3.3 contains a representation of recommendations for the acceleration lane length from four different sources for a highway design speed of 70 mph. These values are all taken from discussions of intersection design. The lengths based on the 1990 AASHTO acceleration rates for passenger cars are also provided.

7. Most of the states that responded to the survey had standards related to the 1990 or 1965 AASHTO Policies.
8. Colorado has volume warrants for acceleration lanes. The warrant takes into account a combination of major highway and turning vehicle design hour volumes. For example, for a posted speed of 40 mph and a turning volume of 45 vph or for posted speeds of 40 to 45 mph and a turning volume of 30 vph, the warrant is met. Wisconsin also has volume warrants, but only for tapered entrances, which have no full-width acceleration lanes.
9. Some states do not use acceleration lanes. Examples of their reasons include that the length required is too long and signals provide a better solution if through traffic does not allow for adequate gaps.
10. The practice of implementing acceleration lanes varies for the different Caltrans districts. Both long and short lanes are used

FIGURE 3.3: Comparison of Acceleration Lane Lengths vs. Design Speed
Acceleration Lane Lengths For Highway Design Speed = 70 mph



*Based on acceleration rates for passenger cars given in the 1990 AASHTO Green Book, page 749. The lengths shown are the distances needed to accelerate from the initial speed to 70 mph.

for left and right turning traffic. Short lanes act as a refuge and longer lanes (up to 1000 ft.) are provided for acceleration.

11. Acceleration lanes of uniform width should be preferably 12 feet wide and a minimum of 10 ft.

4. OPERATIONAL ANALYSIS

The objectives of the operational analysis were:

- (a) To determine guidelines for deciding when to implement acceleration lanes and
- (b) To determine the appropriate length of the acceleration lanes.

The operational analysis consisted of several parts:

- (a) An analysis of delay;
- (b) An analysis of merging characteristics;
- (c) A conflict analysis;
- (d) A speed study and
- (e) A level of service analysis.

The delay, merging characteristics and conflict analyses were all based on video data. Speeds were analyzed from data collected with a radar gun, while the level of service (LOS) analysis was carried out using the methodology outlined in the Highway Capacity Manual (HCM) (3).

In the following sections, the site selection for the operational analysis will be described, followed by a description of the basic approach to the data collection. Next, the data reduction methods and results of the delay, merging characteristics, conflict, speed and LOS analyses will be presented.

4.1 Site Selection

It was originally planned to select sixteen sites for videotaping. The site categories were as follows:

- Four-lane divided, high traffic volume, wide median
- Four-lane divided, high traffic volume, narrow median
- Four-lane divided, low traffic volume, wide median
- Four-lane divided, low traffic volume, narrow median
- Two-lane, high traffic volume
- Two-lane, low traffic volume

In the four-lane divided categories, four sites with acceleration lanes for right-turning traffic and four sites with acceleration lanes for left-turning traffic were to be selected, while in the two-lane categories, two sites with acceleration lanes for right turning traffic were to be included. A control site (without acceleration lanes) was to be selected in each category.

The choices of sites, with regard to the length of the acceleration lane, were to be left until an assessment of available sites could be made.

Since the Caltrans highway inventory and records do not allow for the identification of intersections with acceleration lanes, candidate sites were selected through contact with the Caltrans districts and by

visiting the sites. It should be noted that this was a difficult task, since there are few existing acceleration lanes and finding appropriate sites in all of the different categories proved impossible. After consultation with Caltrans, the following site categories were selected:

- Four-lane divided, wide median, high standard acceleration lane
- Four-lane divided, wide median, low standard acceleration lane
- Four-lane divided, narrow median, high standard acceleration lane
- Four-lane divided, narrow median, low standard acceleration lane
- Two-lane, high standard acceleration lane
- Two-lane, low standard acceleration lane

Sites with acceleration lanes for left and right turning traffic were identified in each category as well as control sites for each category. The definitions used for the two categories of four-lane divided highways are as follows:

Wide median: The median on the major highway has adequate width to allow automobiles to cross the near direction of traffic and wait between the two directions before making a left turn (when no left turn acceleration lane is present). The least width median in this category was six ft. Although six ft. is theoretically not wide enough to allow refuge, observations in the field

indicated that a very large proportion of vehicles used the space in this fashion.

Narrow median: The median on the major highway consists of a barrier or painted lines only. In this case, vehicles did not often use the space between the two directions of traffic as a refuge when making left turns. The widest median encountered in this category was four ft.

Long acceleration lanes were designated as high or low standard. A precise definition of what constitutes a high or low standard acceleration lane was not possible, because of the absence of clear definitions in design standards and also since the distribution of lengths in the field did not yield a clear distinction. An attempt was, however, made to find long and short acceleration lanes in each category and to evaluate the differences in performance. In some cases, very short lanes were encountered that were more likely to be used as pockets to wait for an opportunity to turn left into the traffic on the far side of the intersection. As will be discussed later, there is a clear distinction between the operation of these lanes and others that are several hundred feet longer.

Ultimately, eighteen sites were videotaped. The location (District, County & Postmile), type of intersection, acceleration lane type, intersection type, traffic flow rate and acceleration lane length of these sites are shown in Table 4.1.

TABLE 4.1 : Intersection Characteristics

Classification and Intersection Name	District, County & Postmile of Intersection	Avg. Hwy Speed (AHS)	Type of Intersection	Flow Rate 1991 ADT	Accel. Lane Types - R, L, N	Control Type	Accel. Lane Length (ft.)
4-Lane Wide Median							
High Standard							
Elverta	3 SAC-99-35.37	70	Four Leg	22,750	R	Yield	957
Low Standard							
Castro Valley	4 SCL-101-3.721	65	T	46,500	R	Stop	200
High Standard							
Elverta	3 SAC-99-35.37	70	Four Leg	22,750	L	Stop	600
Low Standard							
Tower	4 NAP-29-3.93	60	T	33,600	L	Stop	118
Control Sites							
Spence	5 MON-101-81.03	65	Four Leg	25,500	N	Stop	***
McCloskey	5 SBT-156-11.94	45	Four Leg	16,400	N	Stop	***
4-Lane Narrow Median							
High Standard							
Blackie	5 MON-101-94.28	70	Four Leg	44,200	R	Stop	600
Low Standard							
Summit - NB	4 SCL-17-0.069	70	T	60,700	R	Stop	60
High Standard							
Black	4 SCL-17-4.451	70	T	64,300	L	Stop	225
Low Standard							
Glenwood	4 SCR-17-10.641	70	T	55,600	L	Stop	132
Control Sites							
Tustin	5 MON-101-96.89	70	Four Leg	48,900	N	Stop	***
Echo Valley	5 MON-101-98.69	70	T	48,900	N	Stop	***
2-Lane							
High Standard							
SR 183	5 MON-1-92.213	70	T	30,800	R	Stop	200
Low Standard							
Cuttings Wharf	4 NAP-121-3.04	65	T	20,200	R	Stop	127
High Standard							
Salinas	5 MON-1-101.04	60	T	30,900	L	Stop	216
Low Standard							
Moss Landing	5 MON-1-95.81	60	T	30,900	L	Stop	76
Control Sites							
Bloomfield	4 SCL-152-14.89	60	T	11,400	N	Stop	***

It should be noted that in addition to the planned sixteen sites, two additional sites were videotaped; one control site in the four-lane wide median category and one control site in the four-lane narrow median category. Traffic flow rates encountered during the surveys at the initial sites were too low to compare with the sites with acceleration lanes.

It should be noted that it would have been better had it been possible to use only four-legged intersections instead of having a mix of T-intersections and four-legged intersections, but a lack of adequate sites made this impossible. The operation of right-turn acceleration lanes could only marginally be affected by through traffic on the minor road and therefore the mix of the two types of intersections should not have an appreciable effect on the results. In the case of left-turn acceleration lanes, the lack of through traffic on the minor road at T-intersections versus the presence of through traffic at four-legged intersections could conceivably make a difference. However, since observation showed that through traffic accounted for only a small portion of the traffic on the minor road, this would not affect the results significantly.

Because of the difficulty experienced in finding sites, with right turn acceleration lanes in the four-lane wide median category and that have high traffic flow rates, Elverta Rd. had to be selected, although it has yield control. In a strict sense, the operation of this acceleration lane is not comparable to those that have stop control preceding the movement. As will be discussed later, very few vehicles came to a

complete stop before turning into the right turn acceleration lane regardless of the stop control. Because of this phenomenon, it was considered useful to retain this site for comparative purposes.

4.2 Overview of Data Collection

The basic data collection consisted of videotaping traffic operations at each site for a period of three hours. One video camera was placed in the position which allowed the best observation of the traffic operations in the acceleration lane and of traffic approaching on the minor road and stopping at the stop bar.

4.3 Delay Analysis

Definitions

For the purpose of the analysis, service delay was defined as the difference between the time that the vehicle reached the stop bar and the time that the vehicle entered the intersection. It should be noted that vehicles using acceleration lanes could encounter some additional delay when entering the main line. This delay was defined as auxiliary delay.

It was impossible to measure auxiliary delay precisely for comparison with intersections without acceleration lanes, since the vehicles may be travelling at less than their desired speed before entering the main line. In order to determine this delay, the desired speed would have to

be known. Consequently, the auxiliary delay was defined as the stopped time in the acceleration lane.

Data Analysis and Results

The following analyses were carried out:

- (a) A comparison was made between the delay experienced at intersections with acceleration lanes and those without acceleration lanes for each category of intersection, turning movement and acceleration lane standard.
- (b) The delay experienced at high standard versus low standard acceleration lanes for each category of intersection and turning movement were compared.
- (c) The decreases in delay due to acceleration lanes for right turn movements versus left turn movements were compared.
- (d) A comparison was made of the decreases in delay due to acceleration lanes by type of highway.
- (e) The benefits of acceleration lanes, in terms of the decrease in delay, were compared with the cost of construction and maintenance.

Subsequent to the analyses, some general observations related to the acceleration lanes are made regarding the traffic operations at the intersections. A summary of the more important conclusions is presented at the end of the chapter.

Analysis Within Categories

Analysis of the data indicated that no service delay or stopped delay was experienced by vehicles at the Elverta site. The probable reasons for this phenomenon could be that there was yield control at this site and that the lane was long enough to provide the vehicles with the opportunity to merge without delay. The average service delay for fifteen minute periods are shown against the corresponding flow rates for the remainder of the categories of intersections and acceleration lanes in Figures 4.1 through 4.11. Weighted average delays are presented in Table 4.2 and percentage decreases in delay in Table 4.3. The percentage decrease was calculated as the delay at the control site minus the delay at the site with acceleration lane, divided by the delay at the control site. The results of the analysis are discussed for each category of intersection and acceleration lane following the title of each figure and the intersections considered for each category.

Four-Lane Wide Median - Right Turn - Low Standard Acceleration

Lane (Figure 4.1):

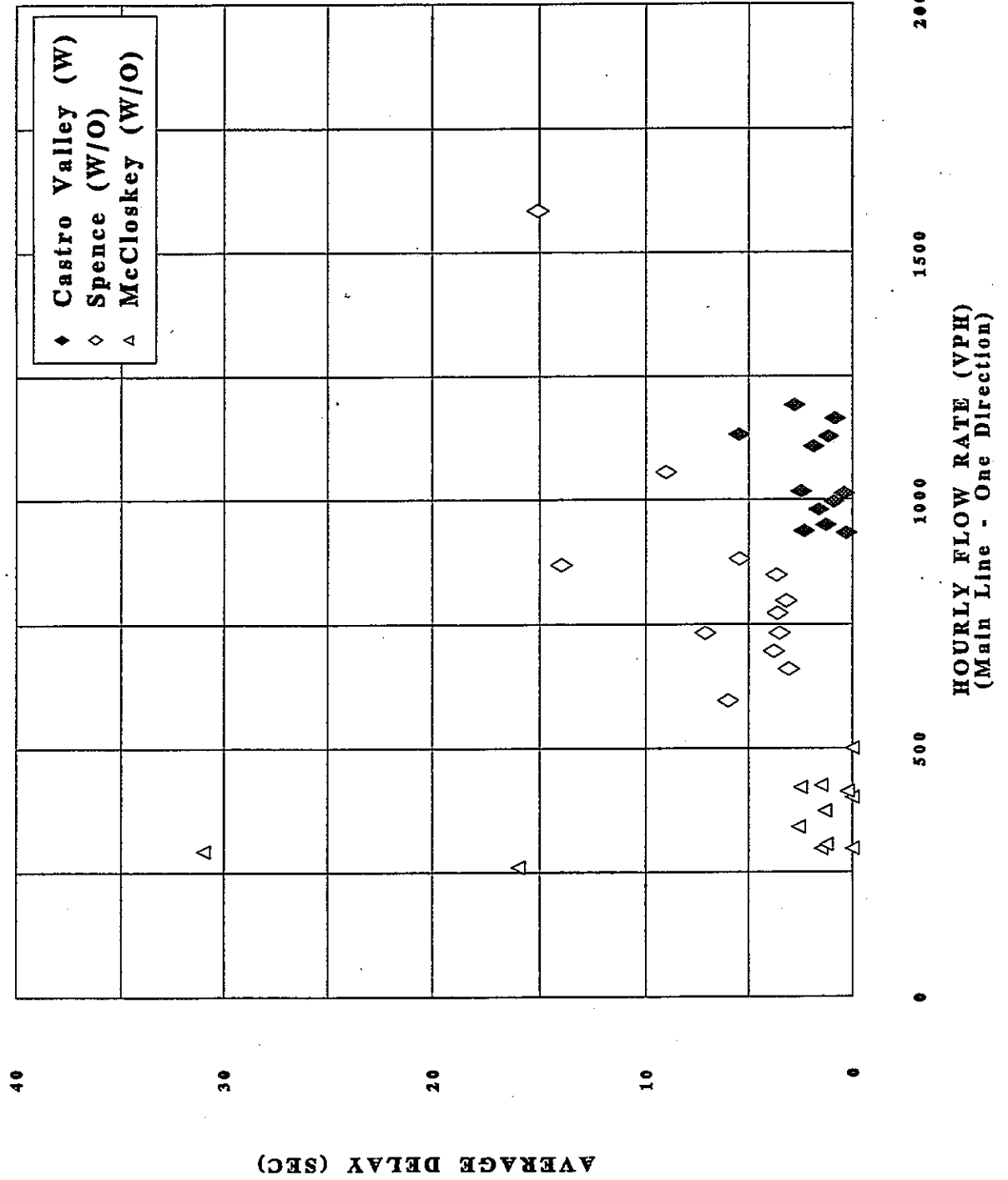
Castro Valley (with low standard acceleration lane)

Spence (without acceleration lane)

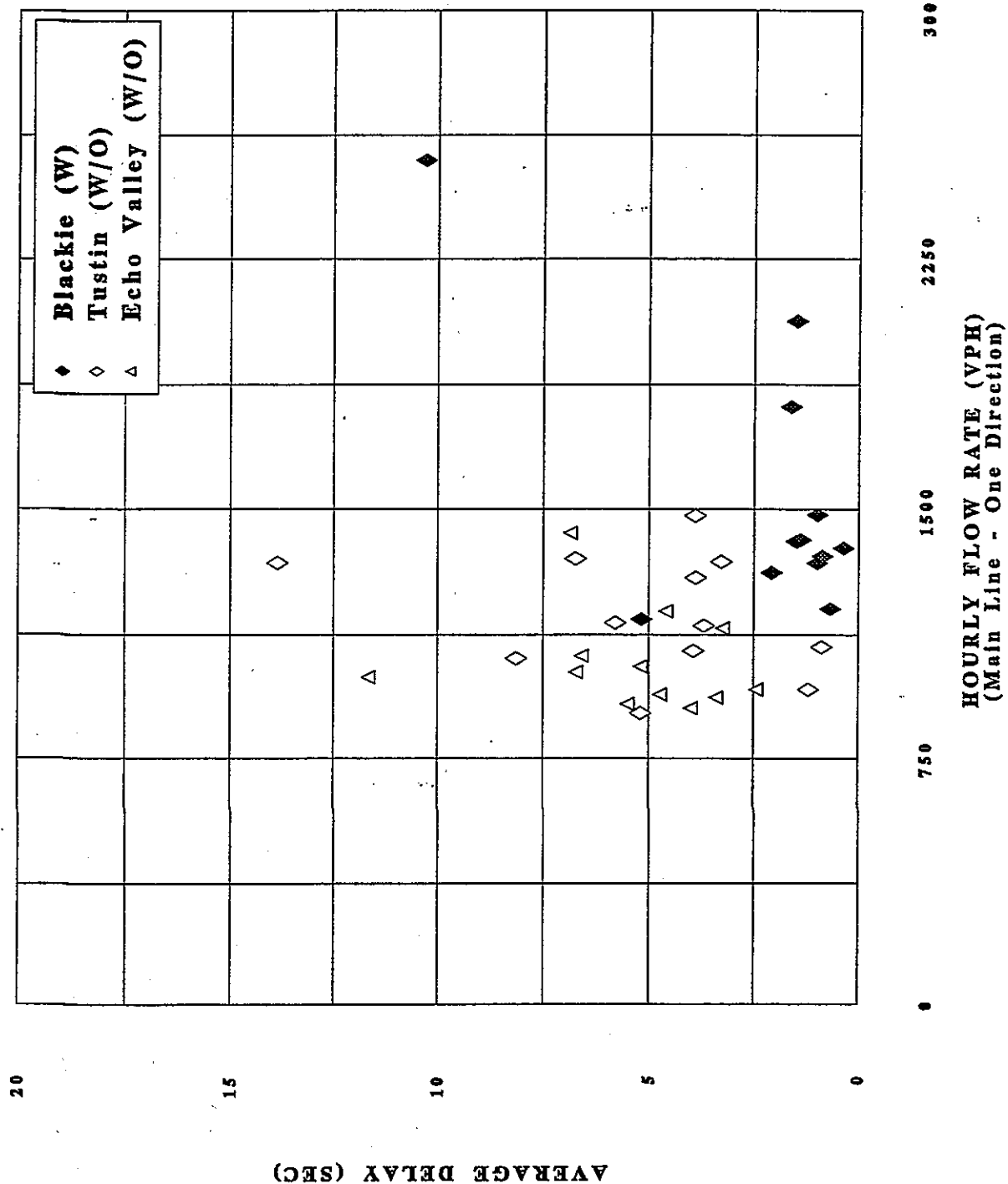
McCloskey (without acceleration lane)

The delay at the Castro Valley intersection is lower than the delay experienced at the McCloskey intersection. During two periods, the delay at the McCloskey intersection was significantly higher than the delay at the Castro Valley intersection. It

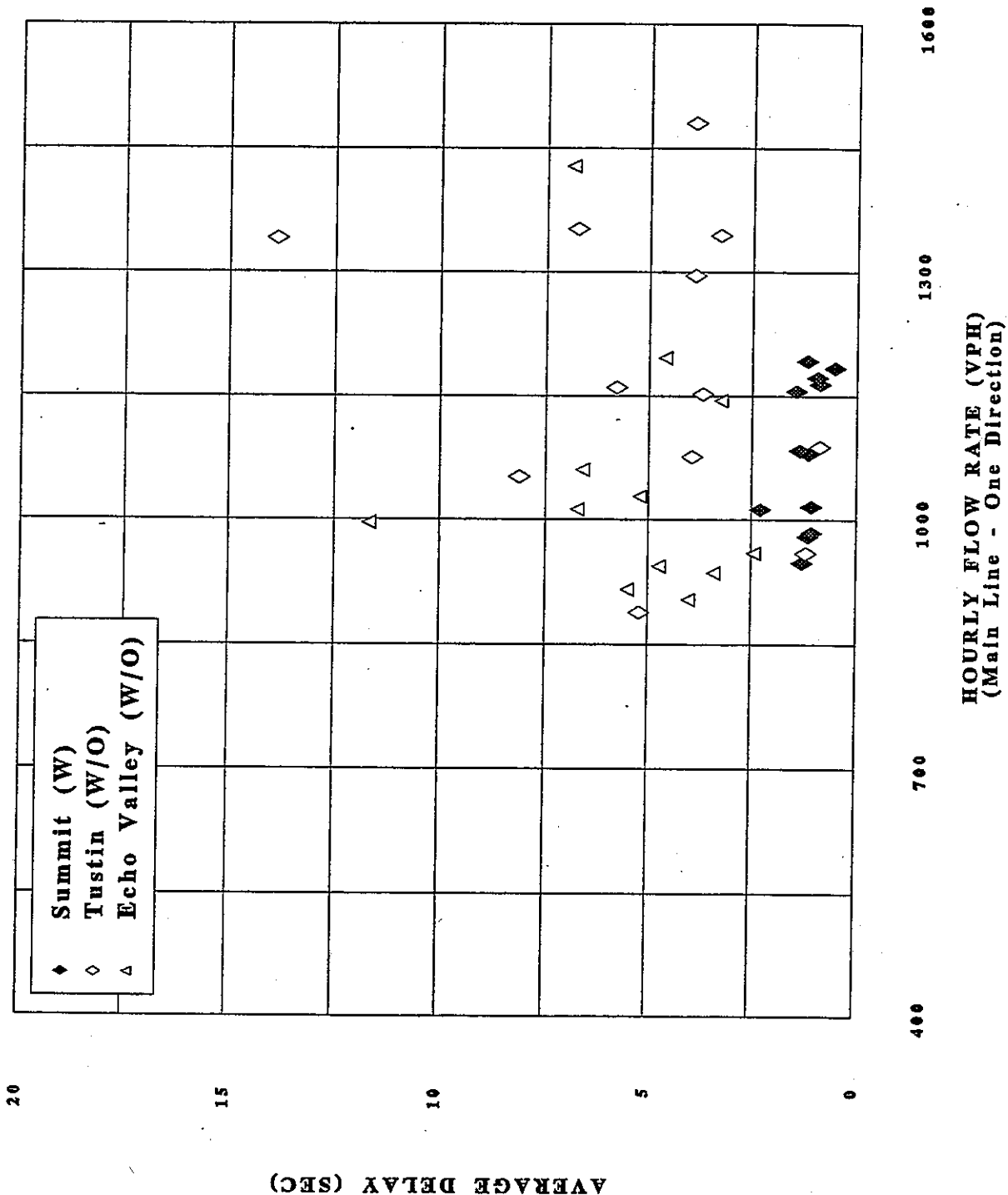
**FIGURE 4.1: Average Delay For Four-Lane Highways - Wide Median - Right Turn
Low Standard Acceleration Lane**



**FIGURE 4.2: Average Delay For Four-Lane Highways - Narrow Median - Right Turn
High Standard Acceleration Lane**



**FIGURE 4.3: Average Delay For Four-Lane Highways - Narrow Median - Right Turn
Low Standard Acceleration Lane**



**FIGURE 4.4: Average Delay For Two-Lane Highways - Right Turn
High Standard Acceleration Lane**

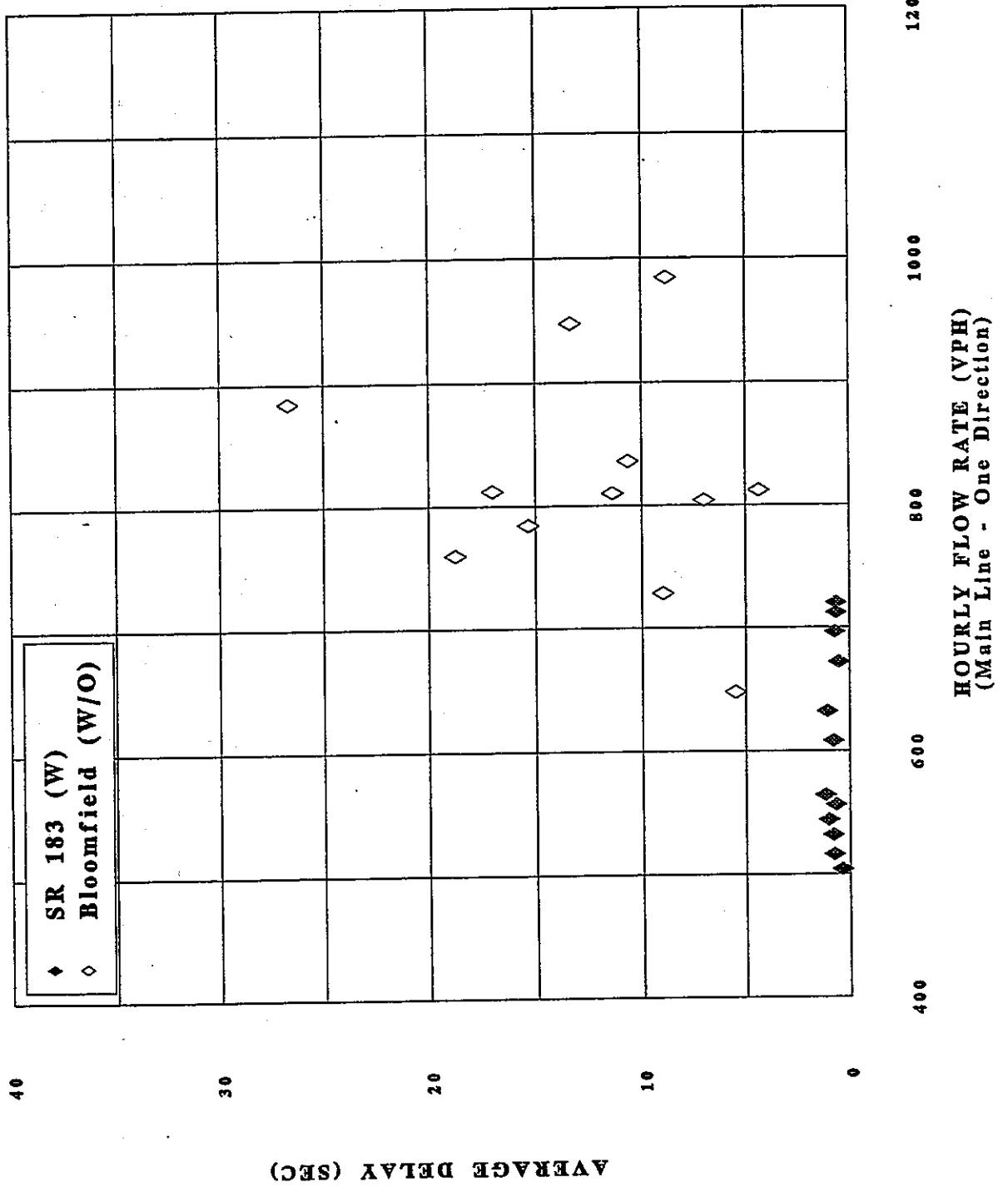
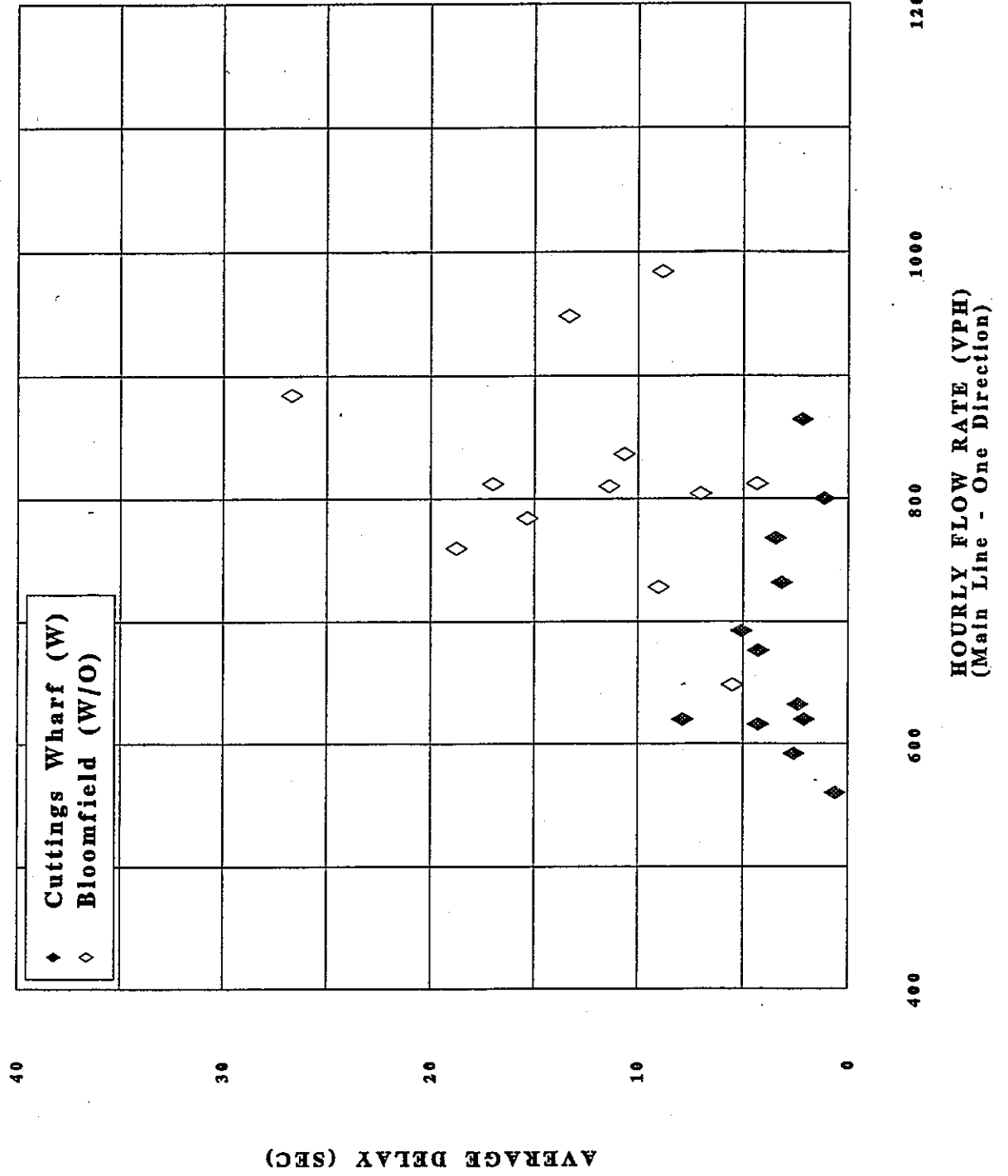
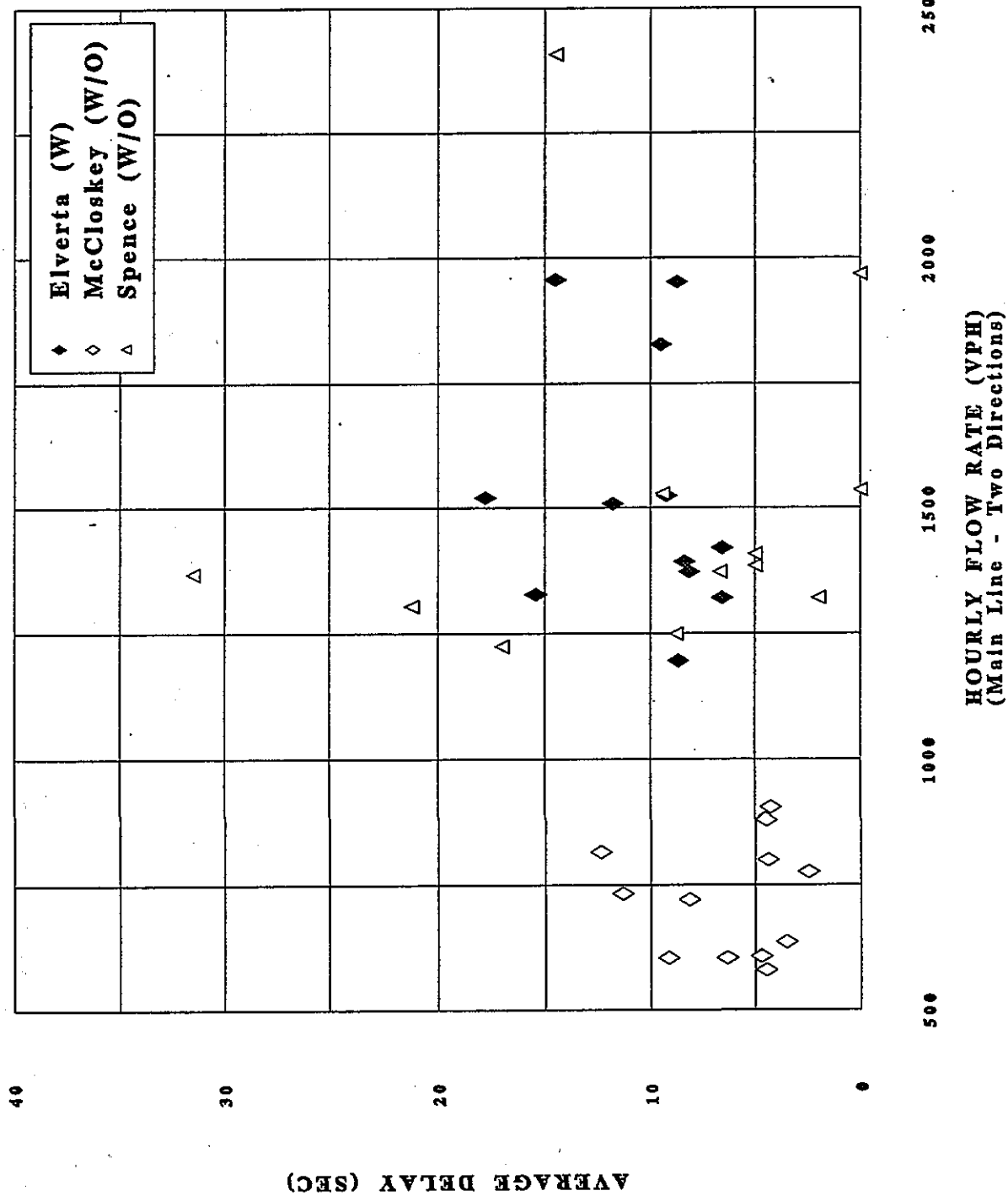


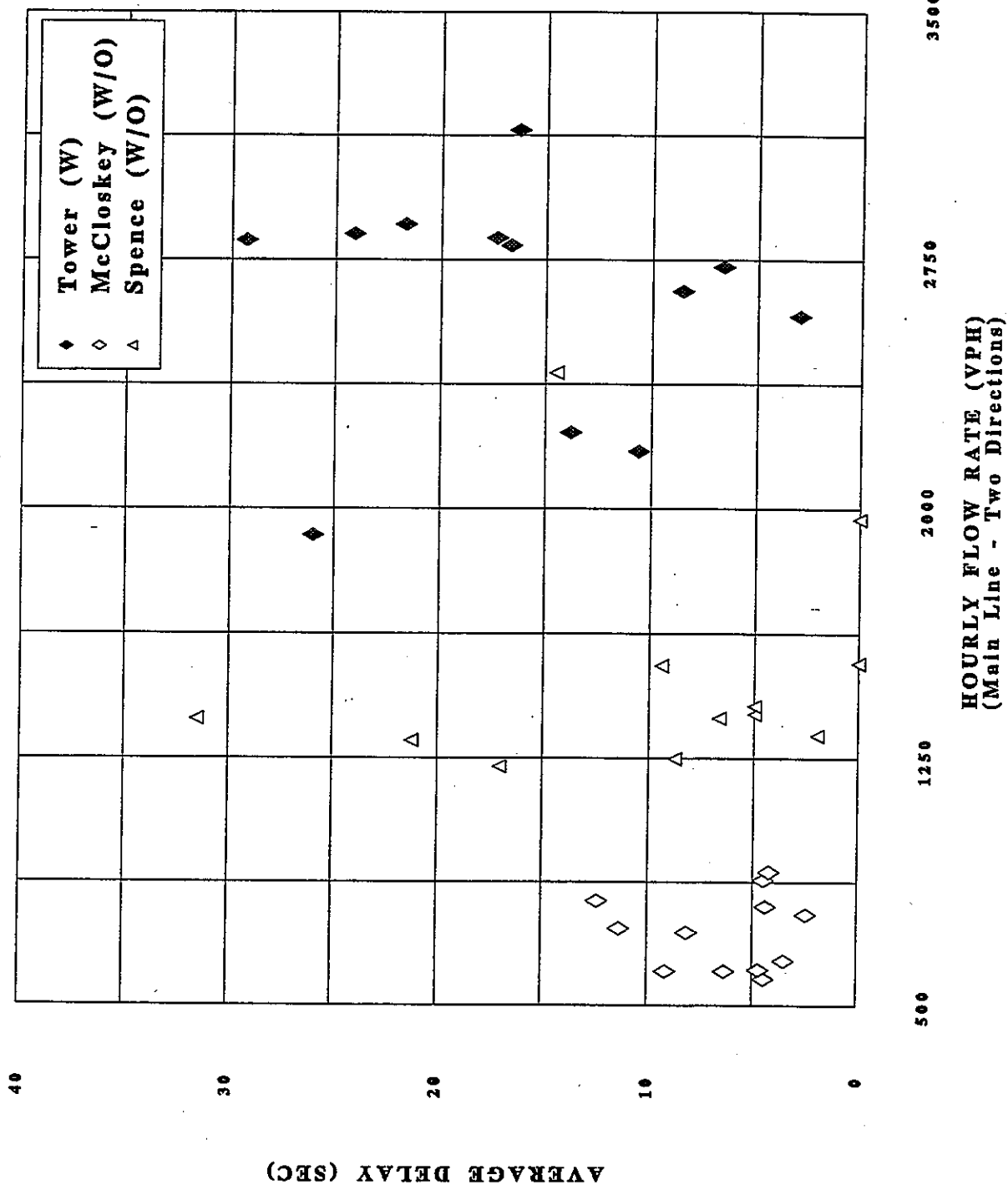
FIGURE 4.5: Average Delay For Two-Lane Highways - Right Turn
Low Standard Acceleration Lane



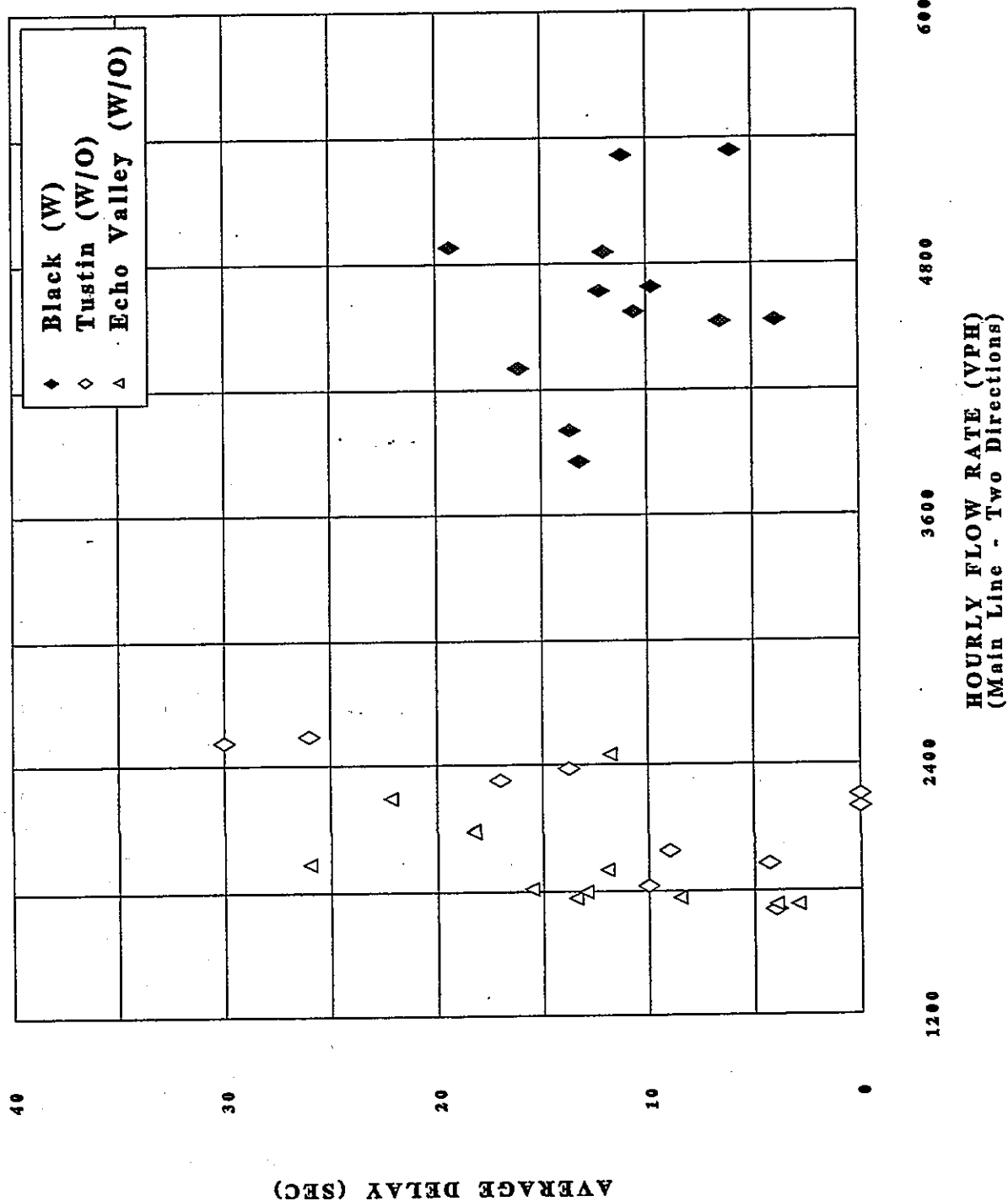
**FIGURE 4.6: Average Delay For Four-Lane Highways - Wide Median - Left Turn
High Standard Acceleration Lane**



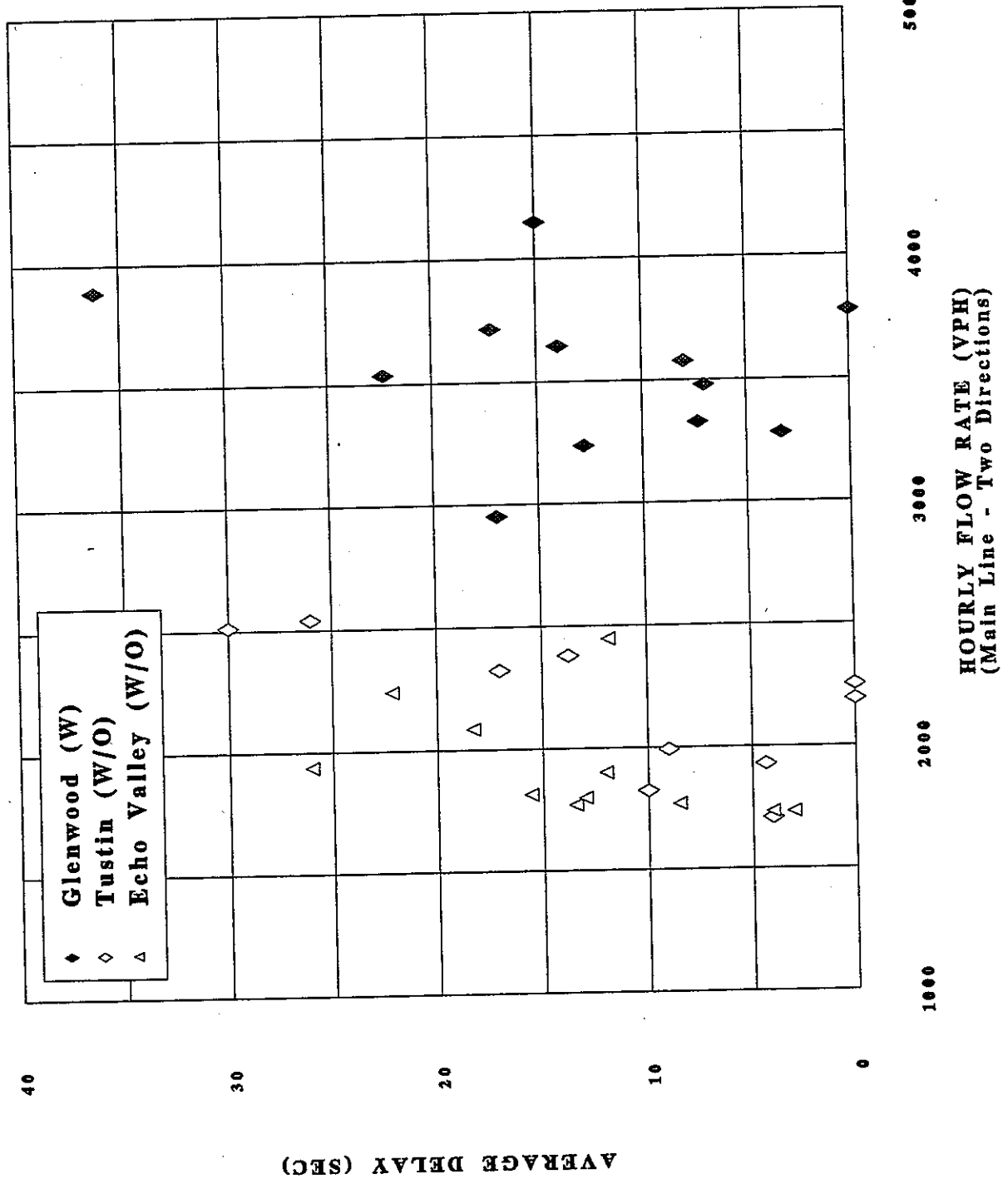
**FIGURE 4.7: Average Delay For Four-Lane Highways - Wide Median - Left Turn
Low Standard Acceleration Lane**



**FIGURE 4.8: Average Delay For Four-Lane Highways - Narrow Median - Left Turn
High Standard Acceleration Lane**



**FIGURE 4.9: Average Delay For Four-Lane Highways - Narrow Median - Left Turn
Low Standard Acceleration Lane**



**FIGURE 4.10: Average Delay For Two-Lane Highways - Left Turn
High Standard Acceleration Lane**

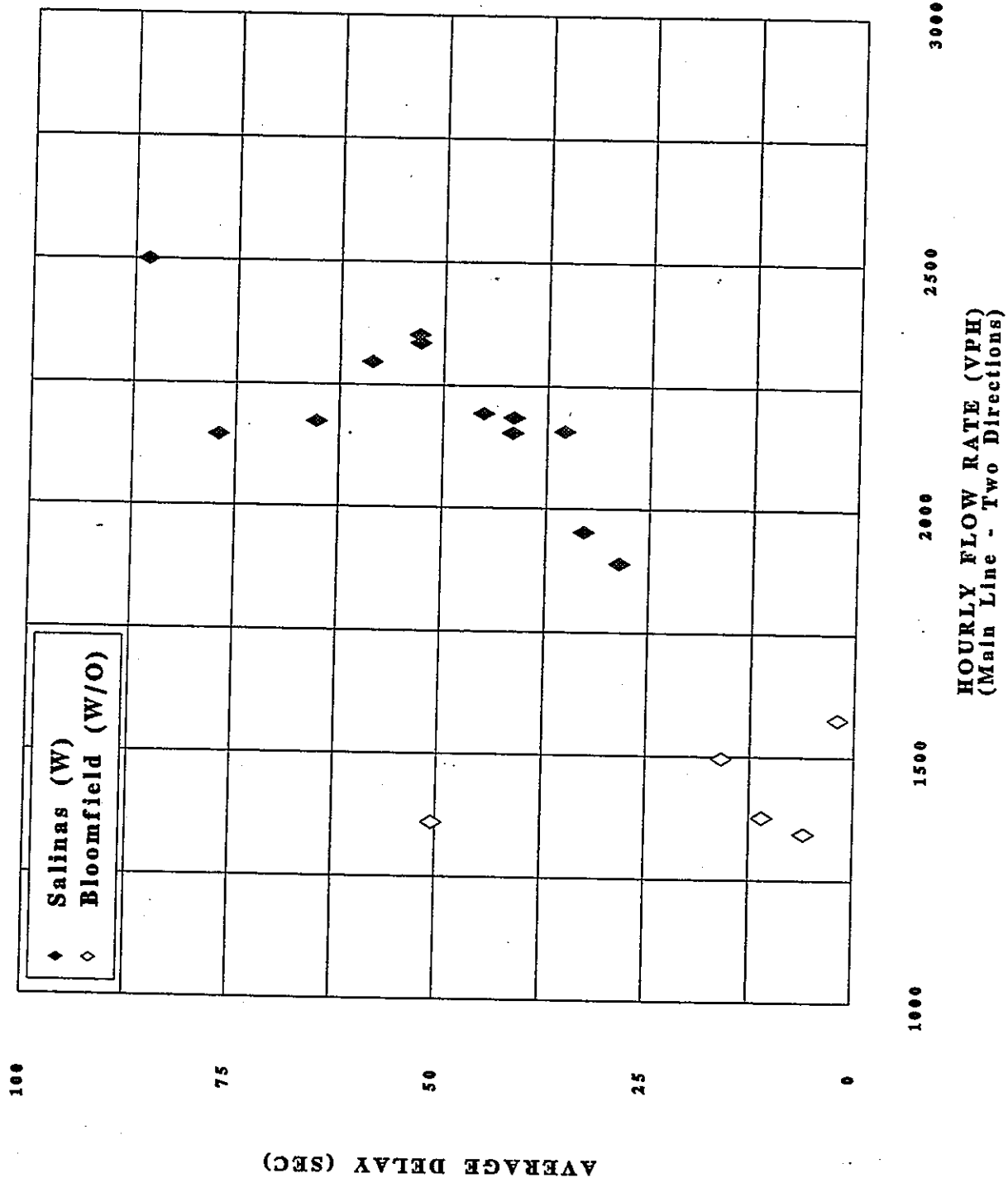


FIGURE 4.11: Average Delay For Two- Lane Highways - Left Turn
Low Standard Acceleration Lane

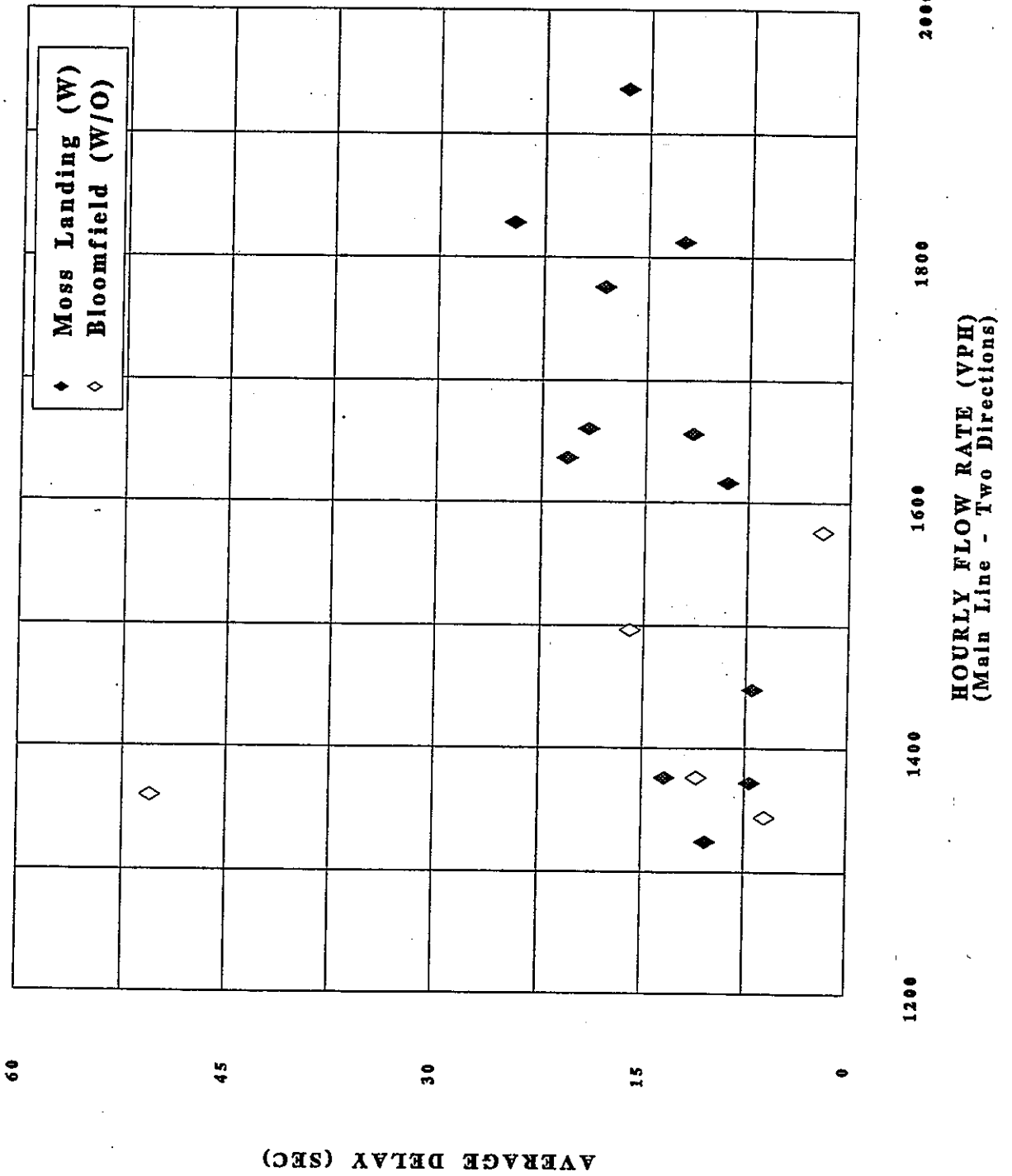


TABLE 4.2 : Average Delay Experienced at Intersections

Classification and Intersection Name	Type of Intersection	Accel. Lane Type - R,L,N	Accel. Lane Length (ft.)	(a) Average Flow Rate (vph) (Major Highway)	(b) Average Delay (sec.)
4-Lane Wide Median					
High Standard					
Elverta	Four Leg	R	957		*
Low Standard					
Castro Valley	T	R	200	(1,045)	(1.79)
High Standard					
Elverta	Four Leg	L	600	1,526	10.44
Low Standard					
Tower	T	L	118	2,627	16.16
Control Sites					
Spence	Four Leg	N	*	1,506 (852)	10.09 (6.45)
McCloskey	Four Leg	N	*	722 (360)	6.31 (4.83)
4-Lane Narrow Median					
High Standard					
Blackie	Four Leg	R	600	(1,541)	(2.29)
Low Standard					
Summit - NB	T	R	60	(1,081)	(1.20)
High Standard					
Black	T	L	225	4,626	11.21
Low Standard					
Glenwood	T	L	132	3,547	13.35
Control Sites					
Tustin	Four Leg	N	*	2,215 (1,182)	9.51 (5.05)
Echo Valley	T	N	*	1,930 (1,044)	12.31 (5.42)
2-Lane					
High Standard					
SR 183	T	R	200	(605)	(0.77)
Low Standard					
Cuttings Wharf	T	R	127	(681)	(3.28)
High Standard					
Salinas	T	L	216	2,194	51.48
Low Standard					
Moss Landing	T	L	76	1,620	14.13
Control Site					
Bloomfield	T	N	*	1,407 (818)	7.13 (12.33)

(a) One-directional flow rate shown in parentheses.

(b) Delay for right turns shown in parentheses.

TABLE 4.3 : Decrease in Delay Resulting from Acceleration Lanes

Classification and Intersection Name	Accel. Lane Length (ft)	Average One-or Two- Directional Flow Rate (vph) (a) Major Highway	Decrease in Average Delay (sec) (b)	Decrease in Average Delay (%) (b)	Compared to	Average One-or Two- Directional Flow Rate (vph) (a) Major Highway
RIGHT TURN						
4-Lane Wide Median						
Low Standard						
Castro Valley	200	(1000)	4.7	72%	Spence	(900)
			3.0	63%	McCloskey	(400)
4-Lane Narrow Median						
High Standard						
Blackie	600	(1500)	2.8	55%	Tustin	(1200)
			3.1	58%	Echo Valley	(1000)
Low Standard						
Summit-NB	60	(1100)	3.9	76%	Tustin	(1200)
			4.2	78%	Echo Valley	(1000)
2-Lane						
High Standard						
SR 183	200	(600)	11.6	94%	Bloomfield	(800)
Low Standard						
Cuttings Wharf	127	(700)	9.0	73%	Bloomfield	(800)
LEFT TURN						
4-Lane Wide Median						
High Standard						
Elverta	600	1500	-0.4	-3%	Spence	1500
			-4.1	-65%	McCloskey	700
Low Standard						
Tower	118	2600	-6.1	-60%	Spence	1500
			-9.9	-156%	McCloskey	700
4-Lane Narrow Median						
High Standard						
Black	225	4600	-1.7	-18%	Tustin	2200
			1.1	9%	Echo Valley	1900
Low Standard						
Glenwood	132	3500	-3.9	-41%	Tustin	2200
			-1.1	-9%	Echo Valley	1900
2-Lane						
High Standard						
Salinas	216	2200	-44.4	-622%	Bloomfield	1400
Low Standard						
Moss Landing	76	1600	-7.0	-98%	Bloomfield	1400

a One - Directional flow rate shown in parentheses

b Decreases shown as positive numbers

should also be noted that the flow rates at the McCloskey intersection were about one third of those found at the Castro Valley intersection. Given equal flow rates, the delay at the McCloskey intersection may be even higher.

The delay at Spence intersection, where the flow rates are closer to those found at McCloskey, was also higher than at the Castro Valley intersection. Again, the delay at the Spence intersection was significantly higher during three periods. These three periods occurred at flow rates in excess of 750 vph for the near direction of traffic.

The lower delay at the intersection with an acceleration lane, even at higher traffic flow rates, indicates that the presence of the acceleration lane does offer an advantage.

Four-Lane Narrow Median - Right Turn - High Standard Acceleration Lane (Figure 4.2):

Blackie (with high standard acceleration lane)

Tustin (without acceleration lane)

Echo Valley (without acceleration lane)

The Blackie intersection exhibited lower delay at comparable flow rates than both the Echo Valley and Tustin intersections, with the exception of one instance, at a high flow rate. There were periods for both of the intersections, without acceleration lanes, wherein the delays were much higher than the norm for the two

intersections. There is, however, no clear indication of a particular flow rate at which this occurs.

Four-Lane Narrow Median - Right Turn - Low Standard Acceleration Lane (Figure 4.3):

Summit (with low standard acceleration lane)

Tustin (without acceleration lane)

Echo Valley (without acceleration lane)

The delay at the Summit intersection was lower at comparable flow rates than the delay at the intersections without acceleration lanes, which indicates that the acceleration lane is beneficial.

Two-Lane - Right Turn - High Standard Acceleration Lane (Figure 4.4):

SR183 (with high standard acceleration lane)

Bloomfield (without acceleration lane)

At comparable flow rates, the delay at the SR183 intersection was consistently lower than at the Bloomfield site. Again, the acceleration lane is beneficial.

Two-Lane - Right Turn - Low Standard Acceleration Lane (Figure 4.5):

Cuttings Wharf (with low standard acceleration lane)

Bloomfield (without acceleration lane)

The delay at the Cuttings Wharf site was lower than at the Bloomfield intersection, but at generally lower flow rates. At comparable flow rates, however, the delay at the Cuttings Wharf site is generally lower, indicating that the acceleration lane decreases delay.

Four-Lane Wide Median - Left Turn - High Standard Acceleration Lane (Figure 4.6):

Elverta (with high standard acceleration lane)
 McCloskey (without acceleration lane)
 Spence (without acceleration lane)

The delay at the Elverta intersection was higher than the delay at the McCloskey intersection. Since the flow rates at the Elverta intersection were higher than the flow rates at the McCloskey intersection, no definite conclusion on the usefulness of the acceleration lane can be drawn.

Slightly higher delay was experienced at the Elverta intersection than at the Spence intersection at comparable flow rates. The higher delay at the Spence intersection during some periods could, however, be evidence that the acceleration lane decreases delay.

Four-Lane Wide Median - Left Turn - Low Standard Acceleration Lane (Figure 4.7):

Tower (with low standard acceleration lane)

McCloskey (without acceleration lane)

Spence (without acceleration lane)

The Tower intersection exhibited higher delay than both the Spence and McCloskey intersections, but at higher flow rates. A conclusion on the benefit of the acceleration lane is therefore impossible.

Four-Lane Narrow Median - Left Turn - High Standard Acceleration Lane (Figure 4.8):

Black (with high standard acceleration lane)

Tustin (without acceleration lane)

Echo Valley (without acceleration lane)

The site with the acceleration lane exhibits slightly lower delay than the Echo Valley site and slightly higher delay than the Tustin site, but at higher flow rates. In the case of the Tustin site, the delay increases when the mainline flow rate is greater than 2300 vph. An acceleration lane may be beneficial at this flow rate.

Four-Lane Narrow Median - Left Turn - Low Standard Acceleration Lane (Figure 4.9):

Glenwood (with low standard acceleration lane)

Tustin (without acceleration lane)

Echo Valley (without acceleration lane)

The delay at the Glenwood intersection is slightly higher than the delay at the Echo Valley intersection and higher than at the Tustin site, but at higher flow rates. As in the case of the high standard acceleration lane, there could be some benefit to having the acceleration lane, but the indication is not as strong.

Two-Lane - Left Turn - High Standard Acceleration Lane

(Figure 4.10):

Salinas (with high standard acceleration lane)

Bloomfield (without acceleration lane)

The delay at the Salinas intersection was substantially higher than at the Bloomfield intersection, but at higher flow rates. No clear conclusion that the acceleration lane was useful could therefore be drawn. It should be noted, however, that the sight distance was restricted for vehicles wanting to use the acceleration lane at the Salinas intersection and this may account for the long delay.

Two-Lane - Left Turn - Low Standard Acceleration Lane

(Figure 4.11):

Moss Landing (with low standard acceleration lane)

Bloomfield (without acceleration lane)

The delay at the Moss Landing intersection was higher than at the control site, but at generally higher flow rates. This, together with the fact that a much higher delay was exhibited at the

Bloomfield intersection during one period, may be an indication that the acceleration lane could be beneficial. The evidence is, however, not clear.

Analysis Of Low Versus High Standard Acceleration Lanes

The information shown in Table 4.2 and 4.3 indicate that for right turn movements, the standard of the acceleration lane did not account for appreciable differences in the percentage decrease in delay for both the four-lane narrow median category and the two-lane category. In fact, the low standard acceleration lanes exhibited larger percentages of decreases in delay. In the four-lane narrow median category, the through traffic flow rate at the Blackie site was substantially higher than at the control site, which may account for the relatively poorer performance of the high standard acceleration lane. Notwithstanding the results, it should be noted that since the actual values of the delay are very small, the difference in the performance of the acceleration lanes are probably not significant.

For left turns, the high standard acceleration lane performed better in the four-lane categories. The difference in the performance of the high standard and low standard acceleration lanes are greater in the wide median than in the narrow median category. A reason for this could possibly be that the difference in the lengths of the two acceleration lanes in the narrow median category is not as great as in the case of the wide median category. In the two-lane high standard acceleration lane category, the sight restrictions at the Salinas site for vehicles

wanting to use the acceleration lane, could account for the poorer performance of the high standard acceleration lane.

Analysis of Right Turn Versus Left Turn Movements

The data presented in Table 4.3 indicate that the acceleration lanes for left turning vehicles generally led to increases in the delay. The reason for this is not obvious. It may be surmised that the drivers use the left turn acceleration lane as a refuge and may stop first before merging with the through traffic even when it is unnecessary to stop. Drivers at intersections without acceleration lanes, on the other hand, generally execute a full turning movement, without stopped delay. This may lead to less total delay, even if drivers are delayed longer at the stop bar. Based on observations of the videotapes, this appears to be a possibility.

It should be noted that the poor performances of the left turn acceleration lanes at the Elverta and Tower sites could be the result of much higher through traffic flow rates as compared to the McCloskey intersection. The higher flow rates make it more difficult to execute the left turn movement. The restricted sight distance at the Salinas site could also account for its poor performance.

Comparison of Performance of Acceleration Lanes by Type of Highway

For right turns, in the four-lane narrow and wide median categories, the percentage decreases shown in Table 4.3 are similar. This is not

surprising, since the width of the median should not influence the right turn movements, except insofar as an overall higher standard of design and a consequent higher standard of operation might be expected at intersections with wide medians.

In the two-lane category, greater benefits may be expected from the acceleration lanes, since through traffic cannot move to another lane to create greater opportunities for merging. The results do not bear this out, however, since the decreases in delay are comparable to those of the four-lane cases. The reason for this could possibly be that the length of the acceleration lanes may not be adequate to decrease the delay.

In the case of left turns, greater benefits could be expected from the acceleration lanes in the narrow median category, as opposed to the wide median category, since turning vehicles cannot use the median for refuge. The results, however, do not bear out this argument.

As in the case of right turns, greater benefits could be expected for two-lane highways. Again, the results do not substantiate this argument. As was discussed before, this could be partly due to the sight restrictions at the Salinas site.

Economic Considerations

A benefit-cost analysis can be used to determine whether the benefits derived from the acceleration lanes exceed the cost of constructing and

maintaining them. Not all benefits and costs are quantifiable in the economic analysis of highways. In practice, however, the benefits and costs are quantified as far as possible and the results of the economic analysis are then used together with qualitative measures to make ultimate decisions regarding the implementation of the proposed project.

In the case of the acceleration lanes, the major benefits would be the decrease in delay and the reduction in accidents. Decreases in vehicle operating costs are usually also included, but this was outside the scope of this project. The costs include the costs of right of way, construction and maintenance.

For the purpose of discussion in this section, only the benefit of reduction in delay was considered. Time savings can be converted to monetary values by multiplying with a monetary rate for each hour or part of an hour saved. The value of time saved depends upon the purpose and the amount of time saved (18). Also, one minute saved for 60 people is not considered equal to 60 minutes saved for one person.

The results of the delay analysis indicate that the time saved is very small, i.e. only a few seconds per vehicle. In terms of only time saved, it would be hard to make a convincing argument that such a few seconds have any value. Nevertheless, delay does not only represent time, but is also a pseudo-measure for aggravation, inconvenience etc. Based on this argument, some value could be attached to the time saved. For the purpose of illustration, an economic analysis was carried out to illustrate what the effect would be if a value were associated with the

time saved.

The following costs for acceleration lanes were obtained from Caltrans:

Pavement: \$15.00/square ft.

This cost was obtained for both left and right turn acceleration lanes. It includes provision for minimal embankment depths (two to four ft.) for right turns and excavation depths of one to two ft. For a 12 ft. wide lane this translates to a cost of \$180/lane-ft.

Maintenance: Flexible pavements: \$1414/lane-mile (\$0.27/lane-ft.)

PCC pavements: \$875/lane-mile (\$0.17/lane-ft.)

Average costs for right of way was unavailable.

The economic feasibility of an acceleration lane will depend upon the number of turning vehicles, the decrease in delay, the time value and the cost. In order to demonstrate the calculation, case studies of the sites in the field were undertaken.

The following assumptions were made:

- ° Useful project life: 20 years
 - ° Real discount rate: 5 percent
 - ° Only construction and maintenance cost taken into account.
 - ° The same decreases in delay existed for six hours per day.
- This assumption does not have any factual support, but an

actual 24 hour count would provide information on whether turning and through traffic flow rates comparable to those experienced during the data collection exist.

- ° Only flexible pavements were considered.
- ° Maintenance costs would remain constant throughout the life of the project. No major rehabilitation would be undertaken during the life of the project.

No value for time was assumed. Instead, the value of time at which the acceleration lane would become economically feasible was calculated.

Left turn acceleration lanes were excluded since they exhibited either no decreases or only very small decreases in delay. The methodology is illustrated for the Castro Valley site and a summary of the analysis and results is presented in Table 4.4.

Castro Valley:

Construction cost = $\$180 * 200 = \$36,000$

Equivalent uniform annual cost = $\$36,000 * 0.08024 = \$2,889$

Annual maintenance cost = $\$0.27 * 200 = \54

Total equivalent uniform annual cost = $\$2,943$

Turning flow rate = 84 vph

Total turning vehicles per year = $84 * 6 * 365 = 183,960$

Average decrease in delay = $(4.7 + 3.0)/2 = 3.9 \text{ sec.}$

Total delay savings = $183,960 * 3.9 = 717,444 \text{ sec} = 199.3 \text{ hours}$

TABLE 4.4: Results of Economic Analysis - Delay (Right Turn Acceleration Lanes)

Category & Name of Intersection	Length (ft.)	Construction Cost	EUAC Construction	Maintenance Cost	Total EUAC	Turning Flow Rate (vph)	Total Turning Veh. per Year	Average Decrease in delay (veh/sec)	Total Delay Savings per Year	Required Time Value
4-Lane Wide										
Castro Valley	200	\$36,000	\$2,889	\$54	\$2,943	84	183,960	3.9	199.3	\$14.80
4-Lane Narrow										
Blackie	600	\$108,000	\$8,666	\$162	\$8,828	42	91,960	3.0	76.7	\$115.80
Summit - SB	60	\$10,800	\$867	\$16	\$883	149	326,310	4.1	371.6	\$2.40
2-Lane										
SR 183	200	\$36,000	\$2,889	\$54	\$2,943	389	851,910	12.0	2839.7	\$1.00
Cuttings Wharf	127	\$10,800	\$1,834	\$34	\$1,868	69	151,110	9.0	377.8	\$4.90

Required time value = $\$2,943/199.3 = \$14.80/\text{veh-hour}$

The results of the analysis indicate that acceleration lanes shorter than 200 ft. could be economically justified if a value of less than \$14.80/hour were to be used and attached to travel time savings of only a few seconds per vehicle. It should be noted that the inclusion of right of way costs would decrease the economic viability.

General Observations

The following general observations were made regarding traffic operations related to turning movements from the cross roads:

- (a) - Some drivers caused long delays at intersections with acceleration lanes due to the fact that they did not appear to know how to use the acceleration lanes. For instance, this was the reason for the long delay experienced by right turning traffic during one period at the Blackie intersection.
- (b) A large number of the vehicles did not come to a complete stop at the stop bar when making right turns into the acceleration lane, even when high through traffic flow rates prevailed. This phenomenon will be discussed in a later section. A few vehicles, making left turns, did not stop at the stop bar.
- (c) At high traffic flow rates, right turning vehicles tended to use the full length of the acceleration lane before merging.

- (d) At sites without acceleration lanes, vehicles sometimes used the shoulder as an acceleration lane when there were few suitable gaps available for merging from a stopped position. For example, at the Tustin intersection, some vehicles travelled up to 200 ft. on the shoulder before merging.
- (e) Vehicles making left turns onto highways, without acceleration lanes, very often stop in the median before merging with the through traffic. This was particularly true when the through traffic flow rate was high. At sites with low flow rates, e.g. the Echo Valley and McCloskey intersections, left turn movements generally did not encounter problems.

4.4 Merging Characteristics

The primary objective of the analysis was to determine whether the acceleration lanes had adequate length to allow for comfortable merging. If merging occurred primarily at the end of the acceleration lane, it could indicate forced merging in the case of long acceleration lanes.

Data Reduction

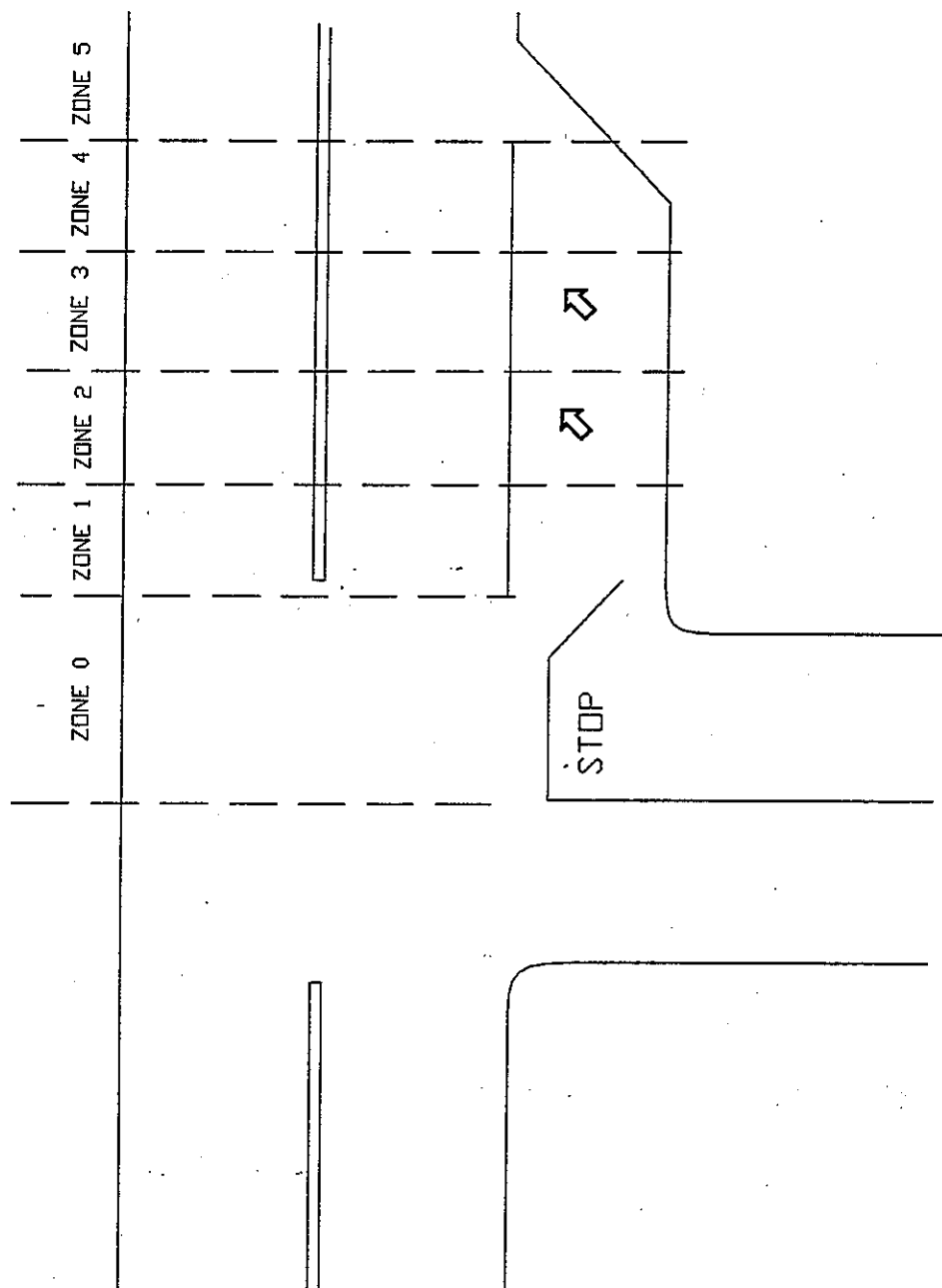
The video data were analyzed to determine the distance along the acceleration lane where the turning vehicles merged with the mainline traffic. The merge location is defined as the point where the wheel, nearest to the through lane, of the merging vehicle, crosses the dividing line between the acceleration lane and the through lane.

For the purpose of the analysis, the distances along the acceleration lane was divided into zones. Zone 0 is defined as the initial zone, measured from the centerline of the cross road to the beginning of the acceleration lane stripe. Zone 5 is the distance beyond the acceleration lane stripe. Zones 1 through 4 are between Zones 0 and 5. Zones 0 through 4 are equal in length. The zone definition is illustrated in Figure 4.12.

In the case of the Elverta site, for the right turn acceleration lane, the distance between the centerline of the cross road and the end of the acceleration lane stripe was divided into five equal distances and designated as zones 0 through 4. The reason for this was that a yield sign controls the right turn movement and the definition of the zones for the remainder of the intersections could not be followed.

Although the data were analyzed for the different types of intersections, the width of the median should not make an appreciable difference in the four-lane category for right turns, except insofar as intersections on highways with wide medians could have generally higher standards. A difference could, however, exist between four-lane and two-lane highways. Through vehicles on two-lane highways do not have the option to give way for a vehicle attempting to merge from the acceleration lane, thereby making the merge more difficult and consequently may require a longer acceleration lane.

During the videotaping sessions, it was observed that a large number of

FIGURE 4.12: Merge Zone Definition

vehicles did not come to a complete stop when turning right into the acceleration lane. Instead, they executed what may be termed a "rolling stop". In order to evaluate this operational phenomenon, an analysis of the number of vehicles not stopping was undertaken.

Data Analysis and Results

Right Turning Vehicles

The numbers and percentages of right turning vehicles merging in each zone are shown in Tables 4.5 through 4.10 for different ranges of through traffic flow rates. The analysis was performed for each 15-minute period. This does not make a difference regarding the number merging in each zone, but does affect the flow rates.

For right turning vehicles on four-lane highways, it appears that the percentages of vehicles, merging near the end of the acceleration lane, are higher for the short acceleration lanes than for the longer acceleration lanes. From the results it may be concluded that an acceleration lane longer than 600 ft. allows adequate opportunity for merging at points along the acceleration lane, whereas merging from the lanes shorter than 200 ft. occurred primarily at the end.

An analysis of the effect of the through traffic flow rate did not show a consistent relationship between through traffic flow rate and percentage merging along the acceleration lane.

**TABLE 4.5: Percentage Merging in Each Zone for Right Turning Vehicles: Four-Lane Highways - Wide Median
High Standard Acceleration Lane (Elverta Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	193	191	191	191	191		957
500 - 1000	0	4	17	10	9	14	54
(%)	0.00	7.41	31.48	18.52	16.67	25.93	100
Total Turning Vehicles	0	4	17	10	9	14	54
Total Percentage	0.00	7.41	31.48	18.52	16.67	25.93	100

**TABLE 4.6: Percentage Merging in Each Zone for Right Turning Vehicles: Four-Lane Highways - Wide Median
Low Standard Acceleration Lane (Castro Valley Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	50	50	50	50		200
500 - 1000	2	6	5	10	3	90	116
(%)	1.72	5.17	4.31	8.62	2.59	77.59	100
1000 - 1500	2	12	5	11	8	98	136
(%)	1.47	8.82	3.68	8.09	5.88	72.06	100
Total Turning Vehicles	4	18	10	21	11	188	252
Total Percentage	1.59	7.14	3.97	8.33	4.37	74.60	100

**TABLE 4.7: Percentage Merging in Each Zone for Right Turning Vehicles: Four-Lane Highways - Narrow Median
High Standard Acceleration Lane (Blackie Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow VPH / Dist. of Accel. Lane	0	150	150	150	150		600
1000 - 1500	0	0	8	16	21	37	82
(%)	0.00	0.00	9.76	19.51	25.61	45.12	100
1500 - 2000	0	0	0	0	3	8	11
(%)	0.00	0.00	0.00	0.00	27.27	72.73	100
2000 - 2500	0	0	0	3	11	19	33
(%)	0.00	0.00	0.00	9.09	33.33	57.58	100
Total Turning Vehicles	0	0	8	19	35	64	126
Total Percentage	0.00	0.00	6.35	15.08	27.78	50.79	100

**TABLE 4.8: Percentage Merging in Each Zone for Right Turning Vehicles: Four-Lane Highways - Narrow Median
Low Standard Acceleration Lane (Summit Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	15	15	15	15		60
500 - 1000	0	2	5	9	16	74	106
(%)	0.00	1.89	4.72	8.49	15.09	69.81	100
1000 - 1500	0	4	18	43	21	256	342
(%)	0.00	1.17	5.26	12.57	6.14	74.85	100
Total Turning Vehicles	0	6	23	52	37	330	448
Total Percentage	0.00	1.34	5.13	11.61	8.26	73.66	100

**TABLE 4.9: Percentage Merging in Each Zone for Right Turning Vehicles: Two-Lane Highways
High Standard Acceleration Lane (SR 183)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	50	50	50	50		200
500 - 1000	10	3	34	40	103	978	1168
(%)	0.86	0.26	2.91	3.42	8.82	83.73	100
Total Turning Vehicles	10	3	34	40	103	978	1168
Total Percentage	0.86	0.26	2.91	3.42	8.82	83.73	100

**TABLE 4.10: Percentage Merging in Each Zone for Right Turning Vehicles: Two-Lane Highways
Low Standard Acceleration Lane (Cuttings Wharf Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	32	32	32	31		127
500 - 1000	0	0	2	14	11	181	208
(%)	0.00	0.00	0.96	6.73	5.29	87.02	100
Total Turning Vehicles	0	0	2	14	11	181	208
Total Percentage	0.00	0.00	0.96	6.73	5.29	87.02	100

In the case of the two-lane highways, the percentages merging near the end of the acceleration lane are approximately equal for the two lengths studied. One reason for this may be that vehicles in the through direction cannot change lanes to allow the vehicle to merge. Another reason may be that the longer lane was not long enough to allow for comfortable merging. Since the percentage merging near the end of the 200 ft. long acceleration lane on the two-lane highway (SR183) is higher than for the 200 ft. long lane on the four-lane highway (Castro Valley), it may be concluded that acceleration lanes on two-lane highways should be longer than on four-lane highways at comparable flow rates.

Left Turning Vehicles

The numbers and percentages of left turning vehicles merging in each zone are shown in Tables 4.11 through 4.16 for different ranges of through traffic flow rates. For four-lane wide median highways it appears that vehicles tended to merge at the end of the acceleration lane for the longer (600 ft.) lane at the Elverta site when the two-directional flow rate exceeded 1500 vph. This could indicate that drivers may find the merge to the right more difficult to contend with than a merge to the left (for right turn lanes), at high flow rates. It may be concluded that left turn acceleration lanes should be longer than right turn acceleration lanes at comparable flow rates.

The same phenomenon was observed at the short (118 ft.) acceleration lane at Tower Road above a two-directional flow rate of 2500 vph. Because of the short length of the acceleration lane, this result should

**TABLE 4.11: Percentage Merging in Each Zone for Left Turning Vehicles: Four-Lane Highways - Wide Median
High Standard Acceleration Lane (Elverta Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	150	150	150	150		600
1000 - 1500	5	5	12	8	2	59	91
(%)	5.49	5.49	13.19	8.79	2.20	64.84	100
1500 - 2000	0	0	2	5	2	50	59
(%)	0.00	0.00	3.39	8.47	3.39	84.75	100
Total Turning Vehicles	5	5	14	13	4	109	150
Total Percentage	3.33	3.33	9.33	8.67	2.67	72.67	100

**TABLE 4.12: Percentage Merging in Each Zone for Left Turning Vehicles: Four-Lane Highways - Wide Median
Low Standard Acceleration Lane (Tower Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	29	29	29	31		118
1500 - 2000	0	0	1	0	0	1	2
(%)	0.00	0.00	50.00	0.00	0.00	50.00	100
2000 - 2500	0	1	3	2	2	14	22
(%)	0.00	4.55	13.64	9.09	9.09	63.64	100
2500 - 3000	0	0	4	2	4	74	84
(%)	0.00	0.00	4.76	2.38	4.76	88.10	100
3000 - 3500	0	0	1	0	0	6	7
(%)	0.00	0.00	14.29	0.00	0.00	85.71	100
Total Turning Vehicles	0	1	9	4	6	95	115
Total Percentage	0.00	0.87	7.83	3.48	5.22	82.61	100

**TABLE 4.13: Percentage Merging in Each Zone for Left Turning Vehicles: Four-Lane Highways - Narrow Median
High Standard Acceleration Lane (Black Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	55	55	55	60		225
3500 - 4000	0	0	1	1	3	13	18
(%)	0.00	0.00	5.56	5.56	16.67	72.22	100
4000 - 4500	2	0	1	0	3	31	37
(%)	5.41	0.00	2.70	0.00	8.11	83.78	100
4500 - 5000	0	1	2	3	25	121	152
(%)	0.00	0.66	1.32	1.97	16.45	79.61	100
5000 - 5500	0	0	0	0	5	51	56
(%)	0.00	0.00	0.00	0.00	8.93	91.07	100
Total Turning Vehicles	2	1	4	4	36	216	263
Total Percentage	0.76	0.38	1.52	1.52	13.69	82.13	100

**TABLE 4.14: Percentage Merging in Each Zone for Left Turning Vehicles: Four-Lane Highways - Narrow Median
Low Standard Acceleration Lane (Glenwood Dr.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	33	33	33	33		132
2500 - 3000	0	0	0	0	1	2	3
(%)	0.00	0.00	0.00	0.00	33.33	66.67	100
3000 - 3500	0	0	3	5	1	3	12
(%)	0.00	0.00	25.00	41.67	8.33	25.00	100
3500 - 4000	0	0	4	5	7	13	29
(%)	0.00	0.00	13.79	17.24	24.14	44.83	100
4000 - 4500	0	0	1	2	2	1	6
(%)	0.00	0.00	16.67	33.33	33.33	16.67	100
Total Turning Vehicles	0	0	8	12	11	19	50
Total Percentage	0.00	0.00	16.00	24.00	22.00	38.00	100

**TABLE 4.15: Percentage Merging in Each Zone for Left Turning Vehicles: Two-Lane Highways
High Standard Acceleration Lane (Salinas Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	40	44	44	44	44		216
1500 - 2000	5	2	0	0	0	22	29
(%)	17.24	6.90	0.00	0.00	0.00	75.86	100
2000 - 2500	3	2	3	9	7	95	119
(%)	2.52	1.68	2.52	7.56	5.88	79.83	100
Total Turning Vehicles	8	4	3	9	7	117	148
Total Percentage	5.41	2.70	2.03	6.08	4.73	79.05	100

**TABLE 4.16: Percentage Merging in Each Zone for Left Turning Vehicles: Two-Lane Highways
Low Standard Acceleration Lane (Moss Landing Rd.)**

Zone Segment of Merge Location	Zone 0	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Total
Thru. Traffic Flow(VPH) / Dist. of Accel. Lane	0	19	19	19	19		76
1000 - 1500	11	14	10	11	3	2	51
(%)	21.57	27.45	19.61	21.57	5.88	3.92	100
1500 - 2000	15	28	16	27	11	7	104
(%)	14.42	26.92	15.38	25.96	10.58	6.73	100
Total Turning Vehicles	26	42	26	38	14	9	155
Total Percentage	16.77	27.10	16.77	24.52	9.03	5.81	100

not be considered very significant. Observations showed that vehicles tended to use the short lane primarily as a refuge while they waited for a suitable gap.

In the four-lane narrow median category, vehicles tended to merge at the end of the longer (225 ft.) acceleration lane at Black Road. The percentage merging at the end increased from 72 percent at a two-directional flow rate of 3500-4000 to 91 percent at a flow rate of 5000-5500 vph. At the 132 ft. acceleration lane at the Glenwood site, vehicles did not merge predominantly at the end, but also used the acceleration lane as a refuge.

Left turning vehicles on the two-lane highways merged at the end of the longer (216 ft.) acceleration lane at the Salinas intersection. This would indicate that the acceleration lane should be longer than 216 ft. for comfortable merging. Again, vehicles used the short (76 ft.) acceleration lane as a refuge and more so than on the four-lane highways.

Vehicles not Stopping

The results of the analysis of the number of vehicles coming to a "rolling stop" are shown in Table 4.17. The magnitude of these numbers may indicate that further consideration of this phenomenon may be warranted.

TABLE 4.17: Total Vehicles Utilizing Rolling Stop vs. Right Turning Vehicles

Classification and Intersection Name	Acceleration Lane Length (ft)	Average One Directional Flow Rate(vph)	Total Turning Vehicles	Number of Vehicles Utilize Rolling Stop	Percent of Vehicles Utilize Rolling Stop
4-Lane Wide Median					
Low Standard					
Castro Valley	200	1000	252	170	67.46
4-Lane Narrow Median					
High Standard					
Blackie	600	1500	126	71	56.35
Low Standard					
Summit-NB	60	1100	448	345	77.01
2-Lane					
High Standard					
SR 183	200	600	1168	908	77.74
Low Standard					
Cuttings Wharf	127	700	208	150	72.12

4.5 Conflict Analysis

It was intended to carry out a conflict analysis at sites with and without acceleration lanes. A conflict was defined as follows (for vehicles either in the acceleration lane or in the through lanes of the main highway):

- (a) A braking maneuver. A braking maneuver was defined as a visible dipping of the front end of the vehicle or brake lights coming on.
- (b) An evasive maneuver.

The analysis proved to be unsatisfactory, since brake lights coming on were not always clear on the videotape, nor was dipping of the front end of the vehicle. Moreover, it was difficult to position the camera in such a way that the brake lights would be visible.

The results indicated, however, that there are in general few conflicts. At most sites there were no conflicts and the largest number observed was three. This conclusion was verified in the field during the videotaping sessions. Conflicts are related to accidents and this absence of conflicts are not unexpected in view of the few accidents that could be directly attributed to acceleration lanes (see Chapter 5 of this report).

At some sites, however, visual observation indicated that there could be more conflicts. These sites included the intersections on SR17 and at

the Salinas intersection.

4.6 Speed Study

The objectives of the speed study were:

- (a) To compare the operating speed of the intersections with acceleration lanes to those without acceleration lanes;
- (b) To compare the maintenance of speed through the intersections with acceleration lanes to those without acceleration lanes and
- (c) To determine the merging speeds to the speeds on the major highway.

Data Collection

A portable radar gun was used to obtain spot speeds of traffic on the major highway, before and after the intersection, and also speeds of traffic turning onto the major highway in the direction of the acceleration lane. The locations at which the speeds were measured varied depending upon whether an acceleration lane was present.

On highways with acceleration lanes, the speeds of 110 vehicles on the major highway were measured before the intersection and after the merge point of the acceleration lane. The speeds of at least 20 vehicles were measured beyond the merge point of the acceleration lane. At sites without acceleration lanes, a similar procedure was followed, with the

exception that the speeds of the through and turning vehicles after the intersection were measured at a point roughly 200 ft. beyond the intersection.

The sites that were studied are the same as those presented in Table 4.1, with the exception that the intersection of Monte Vina and SR17 was substituted for the Summit NB site in the four-lane narrow median category. The Monte Vina site had a 1991 ADT of 64,300. Only one control site in each category was studied.

Data Analysis and Results

The results of the before and after intersection speeds are presented in Table 4.18. There does not appear to be a consistent pattern of differences between the before and after speeds for the same type of highways. It can therefore not be concluded that the presence of an acceleration lane makes a difference in the maintenance of speed through the intersections. There is also not a clear difference between the operating speeds of the intersections with acceleration lanes and those without acceleration lanes. As might be expected, the average speeds decline as the design speed decreases.

A summary of the merging speeds is presented in Table 4.19. The merging speeds are generally comparable, with the exception that the merging speeds from the short acceleration lanes for left turning vehicles in the four-lane categories are substantially lower. The latter phenomenon may be due to the fact that the vehicles use the acceleration lanes as a

TABLE 4.18 : Average Speeds of Main Line Through Traffic Before and After The Intersections

Classification and Intersection Name	Accel. Lane Length (ft.)	(a) Average Speed (mph)	(a) Average Speed (mph)	(b) Standard Deviation	(b) Standard Deviation
		Before	After	Before	After
4-Lane Wide Median					
High Standard					
Elverta					
Low Standard	957	(63.21)	(58.56)	(4.86)	(5.87)
Castro Valley					
High Standard	200	(60.08)	(60.25)	(4.73)	(4.49)
Elverta					
Low Standard	600	63.21	58.56	5.12	5.87
Tower					
Control Sites	118	55.96	56.95	4.45	3.93
Spence					
4-Lane Narrow Median	*	57.08 (58.12)	56.48 (56.09)	5.12 (5.12)	4.80 (5.49)
High Standard					
Blackie					
Low Standard	600	(60.47)	(61.21)	(5.18)	(5.85)
Monte Vina Rd.					
High Standard	60	(53.12)	(55.56)	(5.28)	(5.16)
Black					
Low Standard	225	55.10	50.15	6.25	6.29
Glenwood					
Control Sites	132	51.78	48.32	3.94	3.93
Tustin					
2-Lane	*	53.56 (63.27)	52.06 (61.20)	4.46 (5.10)	4.45 (5.11)
High Standard					
SR 183					
Low Standard	200	(50.70)	(49.67)	(4.74)	(4.31)
Cuttings Wharf					
High Standard	127	(49.29)	(52.66)	(4.58)	(2.90)
Salinas					
Low Standard	216	39.28	47.09	7.22	4.47
Moss Landing					
Control Sites	76	49.79	46.69	6.35	4.24
Bloomfield					
Control Sites	*	56.19 (52.07)	50.09 (54.95)	4.73 (5.09)	4.59 (4.36)

(a) Speed of main line through traffic for right turn acceleration lane shown in parentheses.
 (b) Standard deviation for speeds for right turn acceleration lane shown in parentheses

TABLE 4.19 : Summary of Merging Speeds

Classification and Intersection Name	(a) Speed	(b) Standard Deviation
4-Lane Wide Median		
High Standard	(31.45)	(4.48)
Elverta		(3.75)
Low Standard	(33.40)	
Castro Valley		5.00
High Standard	33.65	
Elverta		5.19
Low Standard	24.00	
Tower		1.92 (2.67)
Control Sites	14.64 (16.90)	
Spence		
4-Lane Narrow Median		
High Standard	(32.95)	(3.05)
Blackie		(6.16)
Low Standard	(29.30)	
Monte Vina Rd.		5.32
High Standard	29.25	
Black		4.30
Low Standard	21.40	
Glenwood		1.35 (4.67)
Control Sites	19.50 (22.45)	
Tustin		
2-Lane		(3.10)
High Standard	(34.10)	
SR 183		(4.07)
Low Standard	(31.67)	
Cuttings Wharf		7.65
High Standard	32.20	
Salinas		3.80
Low Standard	27.95	
Moss Landing		2.43 (5.01)
Control Sites	21.50 (23.00)	
Bloomfield		

(a) Merging speed for right turn acceleration lane shown in parentheses.
 (b) Standard Deviation for right turn acceleration lane shown in parentheses.

refuge and have to accelerate from a stopped condition.

The differences in main line and merging speeds for sites with acceleration lanes are presented in Table 4.20 as well as in Figures 4.13 and 4.14. In general, the speed differentials are larger than expected for the longer acceleration lanes. It could have been expected that the vehicles would have had more opportunity to accelerate before merging.

The speed differentials for two-lane highways are lower than for four-lane highways. This may be due to the fact that vehicles on two-lane highways may have to accelerate more rapidly to merge than on four-lane highways where more gaps are available due to through vehicles changing lanes to allow vehicles to merge. It should be noted, however, that the average speed of the through traffic for two-lane highways is also lower than in the other categories, which would affect the speed difference. The design speed at the Cuttings Wharf site is also lower than in the other categories. This should lower the operating speed and result in lower speed differences. The SR183 site, however, has a 70 mph design speed and also exhibited a small speed differential. It may be concluded that longer acceleration lanes are needed to make acceleration easier.

For left-turning vehicles, the longer acceleration lanes do appear to lead to slightly lower speed differentials than is the case for right turn acceleration lanes, but the differences are not large enough to lead to a definitive conclusion. Again, there is some evidence that the

TABLE 4.20: Comparison Between Main Line (After) and Merging Speed

Classification and Intersection Name	Average Merging Speed (a)	Average Main Line Speed (mph)	Speed Difference
4-Lane Wide Median			
High Standard			
Elverta	(31.45)	(58.56)	27.11
Low Standard			
Castro Valley	(33.40)	(60.25)	26.85
High Standard			
Elverta	33.65	58.56	24.91
Low Standard			
Tower	24.00	56.95	32.95
4-Lane Narrow Median			
High Standard			
Blackie	(32.95)	(61.21)	28.26
Low Standard			
Monte Vina Rd.	(29.30)	(55.56)	26.26
High Standard			
Black	29.25	50.15	20.90
Low Standard			
Glenwood	21.40	48.32	26.92
2-Lane			
High Standard			
SR 183	(34.10)	(49.67)	15.57
Low Standard			
Cuttings Wharf	(31.67)	(52.66)	20.99
High Standard			
Salinas	32.20	47.09	14.89
Low Standard			
Moss Landing	27.95	46.69	18.74
(a) Merging speed for right turn acceleration lane shown in parentheses.			

Figure 4.13: Difference Between Main Line Merging Speeds vs. Acceleration Lane Length - Right Turn

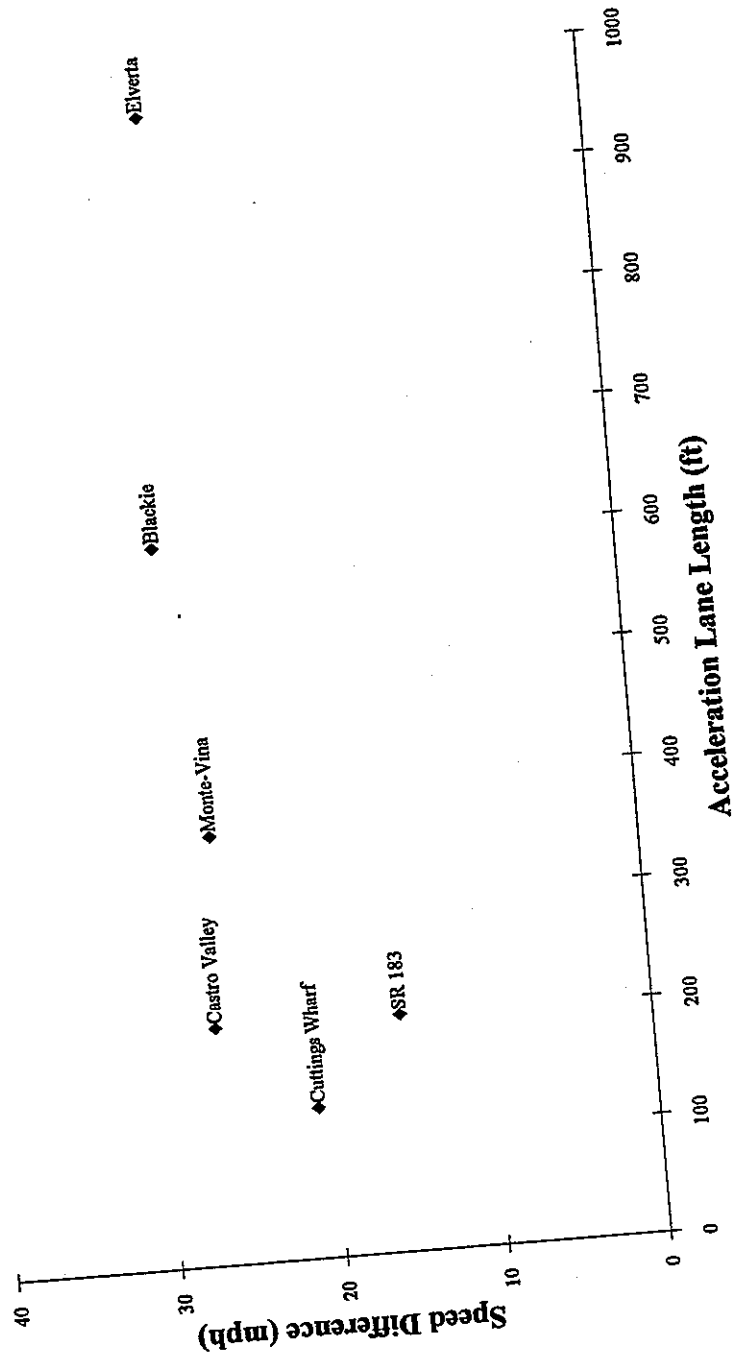
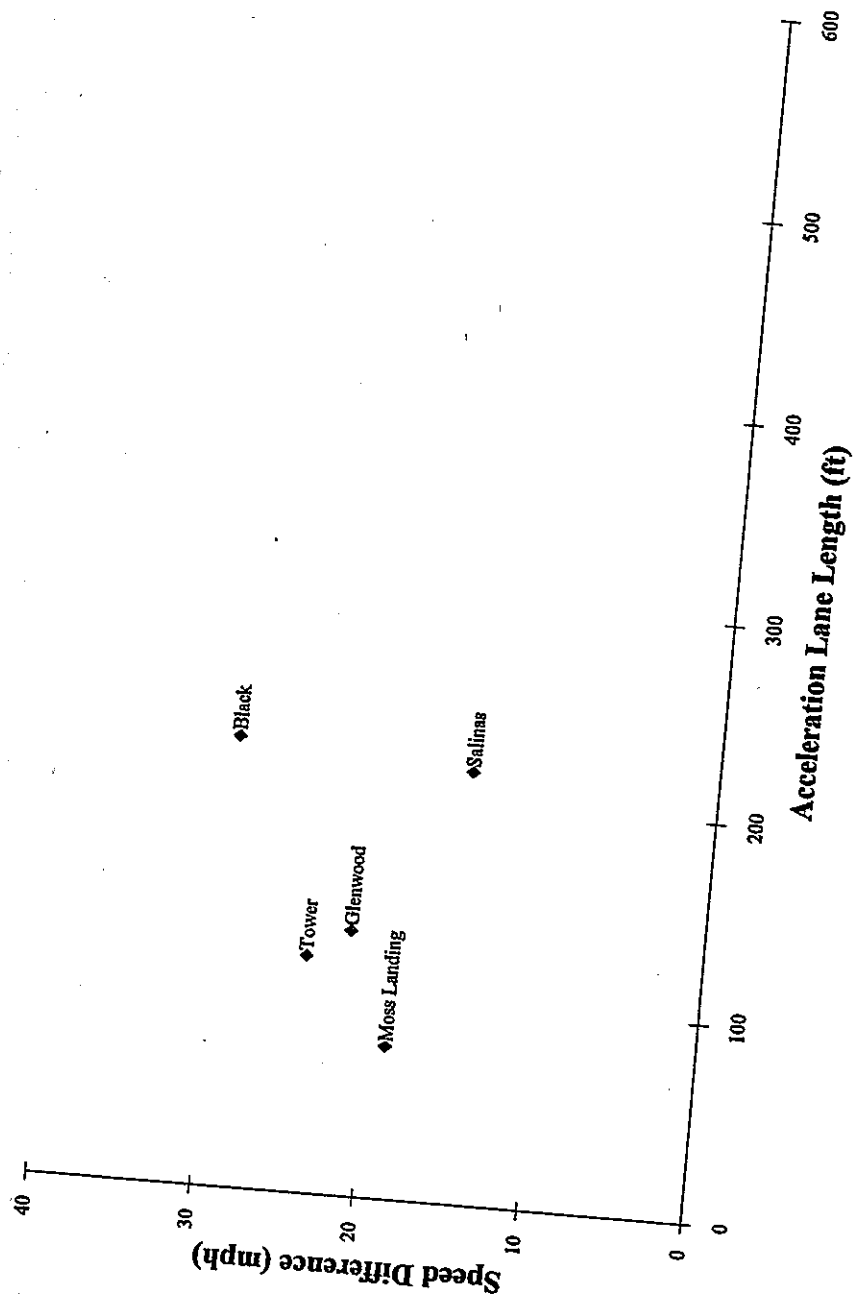


Figure 4.14: Difference Between Main Line Merging Speeds vs. Acceleration Lane Length - Left Turn



speed differentials for two-lane highways are lower.

4.7 Level of Service Analysis

The effects of the presence of an acceleration lane were explored. The determination of the level of service was based on the 1985 Highway Capacity Manual (HCM) method for unsignalized intersections found in Chapter 10 of the HCM.

HCM Methodology

The method employed by the 1985 HCM assumes that major street traffic is not affected by minor street flows. The HCM defines the levels of service for each minor street approach movement and major street left turns according to Table 4.21.

The reserve or unused capacity of a lane is the difference of the shared lane capacity of the lane and the total volume or flow using the lane.

The HCM addresses the addition of an acceleration lane for right turn vehicles only. The existence of the acceleration lane yields a one second reduction in the basic critical gap for the right turning vehicles using the acceleration lane. This reduction in critical gap size for this movement increases the potential or "ideal" capacity for the movement. The increase in potential capacity results in an increase in reserve capacity for the right turning movement and an increase in reserve capacity for the shared lane if other movements do indeed share

TABLE 4.21: Levels of Service for Unsignalized Intersections

RESERVE CAPACITY (PCPH)	LEVEL OF SERVICE	EXPECTED DELAY TO MINOR STREET TRAFFIC
MINOR STREET TRAFFIC		
>/ 400	A	Little or no delay
300-399	B	Short traffic delays
200-299	C	Average traffic delays
100-199	D	Long traffic delays
0-99	E	Very long traffic delays
a	F	a

a

When demand volume exceeds the capacity of the lane extreme delays will be encountered with queuing which may cause severe congestion affecting other traffic movements in the intersection. This condition usually warrants the improvement of the intersection.

Source: 1985 Highway Capacity Manual.

the approach lane with the right turn moment. Therefore, an addition of a right turn acceleration lane increases the reserve capacity and hence improves the level of service, of the right turn movement and does likewise for the shared lane, provided other movements share the approach lane with the right turning vehicles.

LOS Analysis

To determine the effects of the addition of an acceleration lane, calculations were performed using the Highway Capacity Software Version 1.5. From preliminary calculations it was determined that with a base condition, adding an acceleration lane for right turn vehicles does not affect the reserve capacity or level of service of any other movement besides the right turn movement and the shared lane, provided other movements share the approach lane with the right turning vehicles. All of the minor streets observed had single lane approaches, therefore all movements shared the approach with the right turn movement.

Several calculations were performed to determine the change in reserve capacity for the right turning movement and the shared lane when an acceleration lane was introduced to a location where there previously was none. It was also desired to determine these changes under a wide range of potential turning volumes that might occur in the field.

As a beginning point, the average of three hours of observed volume counts were entered for a particular intersection. Then to widen the range of volumes, holding all other volumes constant, one turn volume

count was doubled at a time and then returned to its base value as the next volume was doubled, leaving only one volume differing from its base volume at a time. In some cases where the base volumes were very small, the base volume was multiplied by ten rather than doubled in order to create a larger effect.

After the wide matrix of turning volume conditions were created for the intersection, the level of service was determined for the condition without an acceleration lane for any minor right turn movement and then with an acceleration lane for each minor right turn movement. The change in reserve capacity from the without case to the with case was recorded for the minor right turning movement and its shared lane. This procedure was carried out for four "T" intersections (Echo Valley, HWY 183, Bloomfield, and Cuttings Wharf) and five four-leg intersections (Spence, McCloskey, Monte Vina, Elverta, and Blackie).

Results

Detailed results are contained in Appendix B.

For the "T" intersections, with a sample of 31 with acceleration lane versus without acceleration lane cases, there was an average increase in reserve capacity of 105.6 passenger cars per hour. As can be seen from Table 4.21, an increase of the reserve capacity of 100 vph constitutes one level of service increase. For the shared lane, the reserve capacity improved by an average of only 46.2 passenger cars per hour.

For the four-leg intersections the difference between the with versus without cases, with a sample of 136, had an average increase in reserve capacity of 134.3 passenger cars per hour. For the shared lane, the reserve capacity improved by an average of only 6.6 passenger cars per hour.

4.8 Conclusions

The following major conclusions can be reached for the various analyses performed:

Acceleration Lanes for Right Turning Vehicles

Operational Analysis

The through traffic flow rates (one-directional) and the lengths (excluding the taper) of the acceleration lanes studied, are as follows:

	<u>Flow Rate</u> (vph)	<u>Length</u> (ft.)
Four-Lane divided	1000	200
	1500	600
	1100	60
Two-Lane	600	200
	700	127

Delay Analysis:

1. Acceleration lanes appear to decrease delay for right turning vehicles from the cross road. The measured delay consisted of

delay at the stop bar and stopped delay incurred while attempting to merge. This was true for intersections in the four-lane as well as in the two-lane category.

2. Low standard acceleration lanes exhibited larger percentages of decrease in delay than the longer acceleration lanes in the four-lane categories. In the two-lane category, the high standard acceleration lane performed better.
3. It could be expected that the performance of acceleration lanes at two-lane intersections should be better than those at four-lane intersections, since vehicles at two-lane intersections cannot move over when vehicles want to merge. The results, however, do not bear this out.
4. An economic analysis indicated that acceleration lanes shorter than 200 ft. could be economically justified, if a value of \$14.80 or less were to be assigned to travel time savings of only a few seconds per vehicle. It should be noted that the cost of right of way was not included in the analysis. The only acceleration lane longer than 200 ft., i.e. one with a length of 600 ft., required a travel time value of approximately \$115.00 per hour.

Merging Analysis

1. On four lane-highways, merging appeared to be comfortable for acceleration lanes longer than 600 ft. (for a through traffic flow

rate range of 500 - 2500 vph in one direction). Merging did not appear to be comfortable for acceleration lanes shorter than 200 ft. (for a through traffic flow rate range of 500 - 1500 vph in one direction).

2. Acceleration lanes on two-lane highways should be longer than 200 ft. for comfortable merging (for a through traffic flow rate range of 500 - 1500 vph in one direction).
3. Acceleration lanes should be longer on two-lane highways than on four-lane highways for comparable through traffic flow rates (500 to 1500 vph range).
4. Vehicles often tended to use the shorter lanes (less than 200 ft.) as a refuge to wait for a gap, while the longer lanes are utilized to accelerate in order to merge with the through traffic.
5. A large percentage of vehicles did not come to a complete stop at the stop bar when turning right. Instead, they executed what may be termed a "rolling stop". This phenomenon may warrant further investigation.

Speeds:

1. The presence of acceleration lanes do not appear to affect the average speed upstream and downstream from the intersection.

2. Longer acceleration lanes do not appear to lead to significantly lower differences between through and merging speeds. It may be that the acceleration lanes are not long enough to allow for adequate acceleration or that drivers do not know how to use them.
3. The differences between through and merging speeds appear to be lower for two-lane highways than at four-lane highways.

Level of Service Analysis:

The addition of an acceleration lane for right turning vehicles significantly improves operations for this movement but does not have as great an impact on the shared lane (right turns, left turns and through movements shared this approach) on the cross road nor on the rest of the intersection.

Acceleration Lanes for Left Turning Vehicles

Operational Analysis

The lengths of (excluding taper) and two-directional flow rates at the acceleration lanes studied are as follows:

	<u>Flow Rate</u> (vph)	<u>Length</u> (ft.)
Four-Lane divided	1500	600
	2600	118
	4600	225
	3500	132
Two-Lane	2200	216
	1600	76

Delay Analysis:

1. Acceleration lanes for left turning vehicles do not lead to a significant decrease in delay and is therefore, from this point of view, not economically justified.
2. The high standard acceleration lanes performed better than the low standard lanes in the four-lane categories. In the two-lane category, circumstances at the sites precluded a clear conclusion.
3. Greater benefits could be expected from an acceleration lane in the four-lane narrow median category than in the wide median category, since vehicles cannot use the median as a refuge. The results, however, do not bear this out.
4. Notwithstanding the expectation that greater benefits could be expected from acceleration lanes on two-lane highways than on four-lane highways (since vehicles cannot give way to merging vehicles), the results did not confirm this expectation.

Merging Analysis:

1. For left turning vehicles on four-lane wide median highways, the required length of acceleration lanes for comfortable merging appears to be longer than for a right turning acceleration lane. It appears that the length required for comfortable merging may be

longer than 600 ft. when the two-directional flow rate exceeds 1500 vph.

Vehicles tended to use the short acceleration lanes (less than 118 ft.) as a refuge.

2. In the four-lane narrow median category, the acceleration lane should be longer than 225 ft. to allow for comfortable merging at flow rates higher than 3500 vph. Vehicles used lanes shorter than 132 ft. as a refuge.
3. The length of acceleration lanes for left turning vehicles on two-lane highways should be longer than 216 ft. for comfortable merging above two-directional flow rates of 1500 vph. The acceleration lane with a length of 76 ft. was used as a refuge.
4. Vehicles used the short left turn acceleration lanes more often as a refuge to wait for a gap than in the case of right turns.

Speeds:

1. The presence of acceleration lanes do not appear to affect the average speed upstream and downstream from the intersection.
2. Merging speeds from short acceleration lanes for left turning vehicles appear to be lower than at other acceleration lanes.
3. Longer acceleration lanes do not appear to lead to significantly lower differences between through and merging speeds. It may be

that the acceleration lanes are not long enough to allow for adequate acceleration or that drivers do not know how to use them.

4. The differences between through and merging speeds appear to be lower for two-lane highways than at four-lane highways.

General Observations:

The following general observations were made regarding traffic operations related to turning movements from the cross roads:

1. Some drivers caused long delays at intersections with acceleration lanes due to the fact that they did not appear to know how to use the acceleration lanes.
2. At high traffic flow rates, right turning vehicles tended to use the full length of the acceleration lane before merging.
3. At sites without acceleration lanes, vehicles sometimes used the shoulder as an acceleration lane when there were few suitable gaps available for merging from a stopped position. Some vehicles travelled up to 200 ft. on the shoulder before merging.
4. Vehicles making left turns onto highways, without acceleration lanes, very often stop in the median before merging with the through traffic. This was particularly true when the through traffic flow rate was high.

5. SAFETY ANALYSIS

The possible accident reduction resulting from the presence of an acceleration lane was studied to determine when an acceleration lane should be implemented. For this purpose, the accident rates as well as the accident costs at sites with acceleration lanes were compared with the rates and costs at sites without acceleration lanes.

Initially the analysis was carried out in terms of "avoidable accidents", i.e. accidents that could be directly attributed to the turning maneuver. Since very few of these accidents were found, an additional analysis was carried out. The latter consisted of comparing the rates and costs of all accidents within the intersection zone that encompasses the movements which may be affected by the turning maneuvers.

In order to determine appropriate lengths of acceleration lanes, the performance of different lengths of acceleration lanes were evaluated in terms of accident rates and costs.

The site selection, the approach to the accident analysis, the data reduction and results of the analysis are described in the following sections. Finally, conclusions based on the safety analysis are presented.

5.1 Site Selection

The site selection categories are identical to those identified in Section 4.1. For the purpose of the safety analysis, however, three sites were selected in each category, bringing the total number of sites to 48. The site characteristics are shown in Table 5.1.

5.2 Overview of Data Retrieval

The safety analysis was performed using data from the TASAS data base. The accident reports for the sites were obtained for a three year period i.e. January 1989 through December 1991.

The accidents, which could be directly connected to the presence or absence of an acceleration lane, as used in the initial analysis, were as follows:

Acceleration lanes for vehicles making right turns:

- ° Vehicle from the cross road made a right turn.
- ° Vehicles changed lanes, entered from the shoulder or merged in the same direction as the acceleration lane within the section of highway which contained the acceleration lane.

Acceleration lanes for vehicles making left turns:

- ° Vehicle from the cross road made a left turn.
- ° Vehicles changed lanes, entered from the shoulder or merged in the same direction as the acceleration lane within the

TABLE 5.1 : Intersection Characteristics

Classification and Intersection Name	District, County & Postmile of Intersection	Avg. Hwy Speed (AHS)	Type of Intersection	Flow Rate 1991 ADT	Accel. Lane Types - R, L, N	Control Type	Accel. Lane Length (ft.)
4-Lane Wide Median							
High Standard							
Elverta	3 SAC-99-35.37	70	Four Leg	22,750	R	Yield	957
Catlett	3 SUT-99-7.1	60	Four Leg	19,900	R	Yield	485
Espinosa	5 MON-101-R91.90	70	T	44,200	R	Yield	465
Low Standard							
Castro Valley	4 SCL-101-3.721	65	T	46,500	R	Stop	200
Ocean	7 VEN-101-40.89	70	T	54,500	R	Stop	190
Bay Front	4 SM-84-29.25	50	T	22,750	R	Stop	216
High Standard							
Elverta	3 SAC-99-35.37	70	Four Leg	22,750	L	Stop	600
Catlett	3 SUT-99-7.1	60	Four Leg	14,500	L	Stop	600
Bell Cr.	4 SCL-152-28.8 (Approx.)	55	T	19,500	L	Stop	257
Low Standard							
Tower	4 NAP-29-3.93	60	T	33,600	L	Stop	118
Solano	4 NAP-29-16.63	65	Four Leg	18,600	L	Stop	72
Oak Knoll	4 NAP-29-15.581	65	Four Leg	20,250	L	Stop	60
Control Sites							
Spence	5 MON-101-81.03	65	Four Leg	25,500	N	Stop	***
McCloskey	5 SBT-156-11.94	65	Four Leg	16,400	N	Stop	***
Yerba Buena	5 SLO-1-31.97	65	Four Leg	16,000	N	Stop	***

TABLE 5.1 : Intersection Characteristics

Classification and Intersection Name	District, County & Postmile of Intersection	Avg. Hwy Speed (AHS)	Type of Intersection	Flow Rate 1991 ADT	Accel. Lane Types - R, L, N	Control Type	Accel. Lane Length (ft.)
4-Lane Narrow Median							
High Standard							
Blackie	5 MON-101-94.28	70	Four Leg	44,200	R	Stop	600
Monte-Vina	4 SCL-17-4.62	70	Four Leg	64,300	R	Free RT	360
Pesante	5 MON-101-94.50	70	T	44,200	R	Stop	413
Low Standard							
Summit - NB	4 SCL-17-0.069	70	T	60,700	R	Stop	60
Bay Front	4 SM-84-29.25	50	T	28,500	R	Stop	216
Summit - SB	4 SCL-17-0.069	70	T	60,700	R	Stop	200
High Standard							
Black	4 SCL-17-4.451	70	T	64,300	L	Stop	225
Dunbarton	5 MON-101-100.36	70	Four Leg	48,900	L	Stop	350
Messick #1	5 MON-101-96.39	70	T	48,900	L	Stop	220
Low Standard							
Glenwood	4 SCR-17-10.641	70	T	55,600	L	Stop	132
Sugar Loaf	4 SCR-17-8.712	60	T	55,600	L	Stop	100
Glenwood Cutoff	4 SCR-17-9.071	60	T	55,600	L	Stop	75
Control Sites							
Tustin	5 MON-101-96.89	70	Four Leg	48,900	N	Stop	***
Echo Valley	5 MON-101-98.69	70	T	48,900	N	Stop	***
Crazy Horse	5 MON-101-98.38	70	T	48,900	N	Stop	***

TABLE 5.1 : Intersection Characteristics

Classification and Intersection Name	District, County & Postmile of Intersection	Avg. Hwy Speed (AHS)	Type of Intersection	Flow Rate 1991 ADT	Accel. Lane Types - R, L, N	Control Type	Accel. Lane Length (ft.)
2-Lane							
High Standard							
SR 183	5 MON-1-92.213	70	T	30,800	R	Stop	200
Uvas	4 SCL-152-6.366	55	T	5,000	R	Stop	300
Los Carneros	4 NAP-121-2.79	65	T	20,200	R	Stop	204
Low Standard							
Cuttings Wharf	4 NAP-121-3.04	65	T	20,200	R	Stop	127
Nicolaus	3 SUT-99-11.80	60	T	7,900	R	Yield	160
Madison	4 NAP-29-19.78	55	Four Leg	16,300	R	Stop	58
High Standard							
Salinas	5 MON-1-101.04	60	T	30,900	L	Stop	216
Torero Rd.	4 MON-68-14.09	70	T	19,700	L	Stop	130
Cathedral Oak	4 MON-156-2.37	60	T	22,500	L	Stop	155
Bit Rd.	5 MON-68-9.78	70	T	20,600	L	Stop	125
Molera Rd.	5 MON-1-94.40	60	T	30,900	L	Stop	123
Low Standard							
Moss Landing	5 MON-1-95.81	60	T	30,900	L	Stop	76
Jensen	5 MON-1-99.92	60	T	30,900	L	Stop	85
Meridian	5 MON-156-4.57	60	T	22,500	L	Stop	65
Control Sites							
Bloomfield	4 SCL-152-14.89	60	T	11,400	N	Stop	***
Watsonville	4 SCL-152-5.03	55	T	5,000	N	Stop	***
Espinosa	5 MON-183-R7.65	65	T	16,400	N	Stop	***
Fairview	5 SBT-156-R16.536	65	Four Leg	8,600	N	Stop	***

section of highway which contained the acceleration lane.

For the analysis of all accidents, the following accidents were included:

Acceleration lanes for vehicles making right turns:

All accidents in the same direction as the acceleration lane within the section of highway which contained the acceleration lane. The section was measured from the centerline of the cross road and for a distance beyond the end of the acceleration lane stripe (see Figure 5.1). This distance was selected so as to ensure that all accidents related to the acceleration lane zone were included. As discussed before, the precise ends of the acceleration lanes were hard to determine.

Acceleration lanes for vehicles making left turns:

All accidents which occurred in the intersection area between the edges of the cross road plus all accidents in the same direction as the acceleration lane within the section of highway which contained the acceleration lane. The section was measured from the beginning and, as in the case of the right turn acceleration lanes, a distance beyond the end of the acceleration lane stripe (see Figure 5.2).

In the case of the control sites (without acceleration lanes), the lengths over which the accident rates were compared with the sites with

FIGURE 5.1: Area of Accident Data - Right Turn

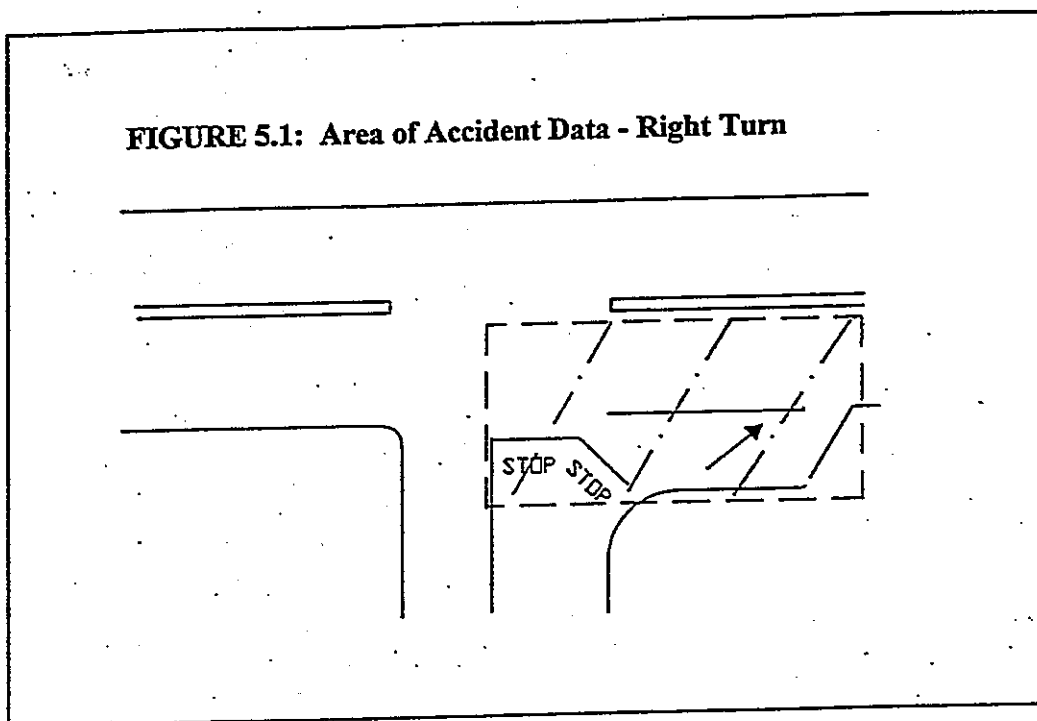
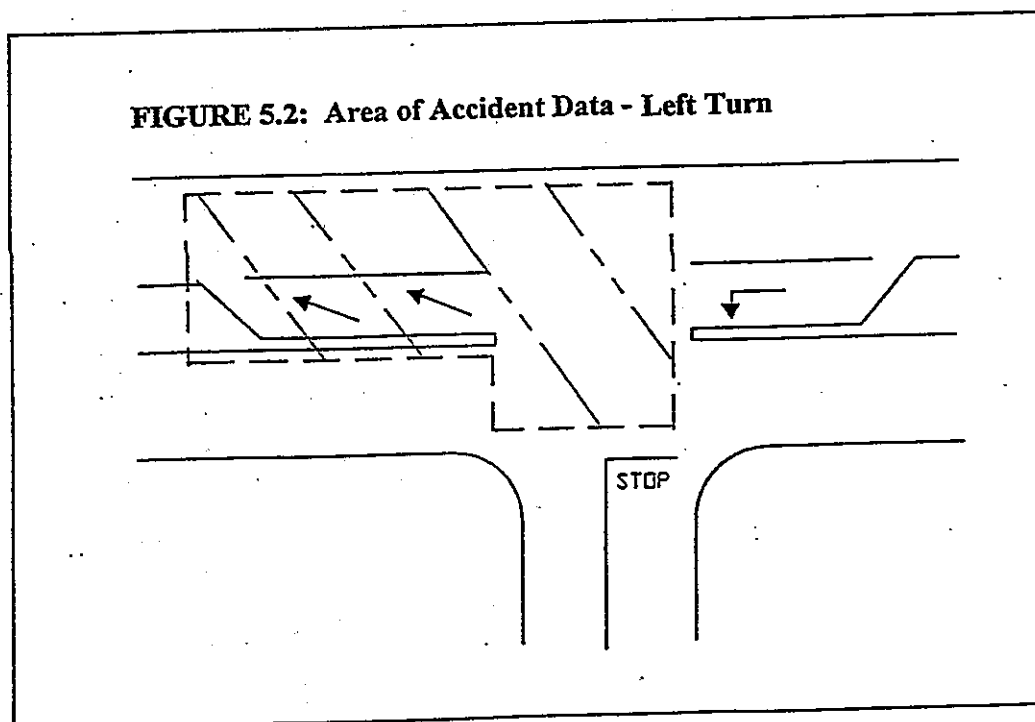


FIGURE 5.2: Area of Accident Data - Left Turn



acceleration lanes, were the same as the length for the site with the acceleration lane.

5.3 Data Analysis and Results

Accident Rates and Costs

The results of the analysis for the "avoidable" accidents are shown in Tables 5.2 through 5.11. The rates are expressed in accidents per million vehicles. The analysis was carried out for all types of accidents and for just injury plus fatal accidents. No fatal accidents were found.

In each case the percentage decrease in accident rate was based on the comparison between a site with an acceleration lane and the control site. The results are considered to be inconclusive because of the low number of accidents found. For this reason, an accident cost analysis was not carried out.

Tables 5.12 through 5.21 contain the results of the accident rate analysis for all accidents in the zones described in the previous section. Average accident rates were calculated according to sites without acceleration lanes and also according to sites with acceleration lanes. These averages are presented in Tables 5.22 through 5.24. It should be noted that the averages are weighted according to vehicle volume. The results of the accident cost analysis are shown in Tables 5.25 through 5.34. Averages are shown in Tables 5.35 through 5.37.

TABLE 5.2: Comparison of Accident Rates for Four-Lane Wide Median Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Spence / Elverta	13.87/12.05	0/0	0.00	0.00		0/0	0.00	0.00	
Spence / Callett	13.87/10.11	0/0	0.00	0.00		0/0	0.00	0.00	
Spence / Espinosa	13.87/24.22	0/0	0.00	0.00		0/0	0.00	0.00	
Spence / Castro Valley	13.87/24.09	0/0	0.00	0.00		0/0	0.00	0.00	
Spence / Ocean	13.87/29.84	0/0	0.00	0.00		0/0	0.00	0.00	
Spence / Bay Front	13.87/17.06	0/0	0.00	0.00		0/0	0.00	0.00	
Left Turn									
Spence / Elverta	27.74/24.09	0/1	0.00	0.08		0/1	0.00	0.08	
Spence / Callett	27.74/19.49	0/0	0.00	0.00		0/0	0.00	0.00	
Spence / Bell Cr.	27.74/18.82	0/1	0.00	0.05		0/0	0.00	0.00	
Spence / Tower	27.74/37.16	0/2	0.00	0.05		0/0	0.00	0.00	
Spence / Solano	27.74/20.88	0/0	0.00	0.00		0/0	0.00	0.00	
Spence / Oak Knoll	27.74/21.48	0/4	0.00	0.19		0/1	0.00	0.05	

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.3: Comparison of Accident Rates for Four-Lane Wide Median Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
McCloskey / Elverta	7.37/12.05	0/0	0.00	0.00		0/0	0.00	0.00	
McCloskey / Catlett	7.37/10.11	0/0	0.00	0.00		0/0	0.00	0.00	
McCloskey / Espinosa	7.37/24.22	0/0	0.00	0.00		0/0	0.00	0.00	
McCloskey / Castro Valley	7.37/24.09	0/0	0.00	0.00		0/0	0.00	0.00	
McCloskey / Ocean	7.37/29.84	0/0	0.00	0.00		0/0	0.00	0.00	
McCloskey / Bay Front	7.37/17.06	0/0	0.00	0.00		0/0	0.00	0.00	
Left Turn									
McCloskey / Elverta	14.75/24.09	4/1	0.27	0.08	-70.37	1/1	0.07	0.08	14.29
McCloskey / Catlett	14.75/19.49	4/0	0.27	0.00	-100	1/0	0.07	0.00	-100
McCloskey / Bell Cr.	14.75/18.82	4/1	0.27	0.05	-81.48	1/0	0.07	0.00	-100
McCloskey / Tower	14.75/37.16	4/2	0.27	0.05	-81.48	1/0	0.07	0.00	-100
McCloskey / Solano	14.75/20.88	4/0	0.27	0.00	-100	1/0	0.07	0.00	-100
McCloskey / Oak Knoll	14.75/21.48	4/4	0.27	0.19	-29.63	1/1	0.07	0.05	-28.57

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.4: Comparison of Accident Rates for Four-Lane Wide Median Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Yerba Buena / Elverta	8.60/12.05	0/0	0.00	0.00		0/0	0.00	0.00	
Yerba Buena / Callitt	8.60/10.11	0/0	0.00	0.00		0/0	0.00	0.00	
Yerba Buena / Espinosa	8.60/24.22	0/0	0.00	0.00		0/0	0.00	0.00	
Yerba Buena / Castro Valley	8.60/24.09	0/0	0.00	0.00		0/0	0.00	0.00	
Yerba Buena / Ocean	8.60/29.84	0/0	0.00	0.00		0/0	0.00	0.00	
Yerba Buena / Bay Front	8.60/17.06	0/0	0.00	0.00		0/0	0.00	0.00	
Left Turn									
Yerba Buena / Elverta	17.19/24.09	2/1	0.12	0.08	-33.33	0/1	0.00	0.08	
Yerba Buena / Callitt	17.19/19.49	2/0	0.12	0.00	-100	0/0	0.00	0.00	
Yerba Buena / Bell Cr.	17.19/18.82	2/1	0.12	0.05	-58.33	0/0	0.00	0.00	
Yerba Buena / Tower	17.19/37.16	2/2	0.12	0.05	-58.33	0/0	0.00	0.00	
Yerba Buena / Solano	17.19/20.88	2/0	0.12	0.00	-100	0/0	0.00	0.00	
Yerba Buena / Oak Knoll	17.19/21.48	2/4	0.12	0.19	58.33	0/1	0.00	0.05	

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.5: Comparison of Accident Rates for Four-Lane Narrow Median Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Tustin / Blackie	25.49/24.31	1/0	0.04	0.00	-100	1/0	0.04	0.00	-100
Tustin / Monte-Vina	25.49/36.46	1/1	0.04	0.03	-25	1/1	0.04	0.03	-25
Tustin / Pesante	25.49/24.31	1/1	0.04	0.04	0	1/1	0.04	0.04	0
Tustin / Summit - NB	25.49/32.07	1/11	0.04	0.34	750	1/3	0.04	0.09	125
Tustin / Bay Front	25.49/17.06	1/0	0.04	0.00	-100	1/0	0.04	0.00	-100
Tustin / Summit - SB	25.49/32.07	1/1	0.04	0.03	-25	1/1	0.04	0.03	-25
Left Turn									
Tustin / Black	50.97/72.45	2/5	0.04	0.01	-75	1/3	0.02	0.06	200
Tustin / Dunbarton	50.97/50.52	2/0	0.04	0.00	-100	1/0	0.02	0.00	-100
Tustin / Messick #1	50.97/50.97	2/4	0.04	0.08	100	1/4	0.02	0.08	300
Tustin / Glenwood	50.97/58.25	2/9	0.04	0.15	275	1/6	0.02	0.10	400
Tustin / Sugar Loaf	50.97/58.25	2/1	0.04	0.02	-50	1/0	0.02	0.00	-100
Tustin / Glenwood Cutoff	50.97/58.25	2/1	0.04	0.02	-50	1/0	0.02	0.00	-100

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.6: Comparison of Accident Rates for Four-Lane Narrow Median Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Echo Valley / Blackie	25.49/24.31	1 / 0	0.04	0.00	-100	0 / 0	0.00	0.00	
Echo Valley / Monte-Vina	25.49/36.46	1 / 1	0.04	0.34	750	0 / 3	0.00	0.09	
Echo Valley / Pesante	25.49/24.31	1 / 1	0.04	0.04	0	0 / 1	0.00	0.04	
Echo Valley / Summit - NB	25.49/32.07	1 / 11	0.04	0.03	-25	0 / 1	0.00	0.03	
Echo Valley / Bay Front	25.49/17.06	1 / 0	0.04	0.00	-100	0 / 0	0.00	0.00	
Echo Valley / Summit - SB	25.49/32.07	1 / 1	0.04	0.03	-25	0 / 1	0.00	0.03	
Left Turn									
Echo Valley / Black	50.97/72.45	5 / 5	0.10	0.10	0	3 / 3	0.06	0.06	0
Echo Valley / Dunbarton	50.97/50.52	5 / 0	0.10	0.00	-100	3 / 0	0.06	0.00	-100
Echo Valley / Messick #1	50.97/50.97	5 / 4	0.10	0.80	700	3 / 4	0.06	0.08	33
Echo Valley / Glenwood	50.97/58.25	5 / 9	0.10	0.15	50	3 / 6	0.06	0.10	66.67
Echo Valley / Sugar Loaf	50.97/58.25	5 / 1	0.10	0.02	-80	3 / 0	0.06	0.00	-100
Echo Valley / Glenwood C.	50.97/58.25	5 / 1	0.10	0.02	-80	3 / 0	0.06	0.00	-100

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.8: Comparison of Accident Rates for Two-Lane Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Bloomfield / SR 183 & 1	5.82/15.57	0 / 2	0.00	0.13		0 / 2	0.00	0.13	
Bloomfield / Uvas	5.82/3.08	0 / 0	0.00	0.00		0 / 0	0.00	0.00	
Bloomfield / Los Carneros	5.82/10.13	0 / 0	0.00	0.00		0 / 0	0.00	0.00	
Bloomfield / Cuttings Wharf	5.82/10.95	0 / 0	0.00	0.00		0 / 0	0.00	0.00	
Bloomfield / Nicolaus	5.82/4.29	0 / 0	0.00	0.00		0 / 0	0.00	0.00	
Bloomfield / Madison	5.82/8.60	0 / 1	0.00	0.12		0 / 0	0.00	0.00	
Left Turn									
Bloomfield / Salinas	11.64/31.17	6 / 15	0.52	0.48	-7.69	4 / 8	0.34	0.26	-23.53
Bloomfield / Torero	11.64/21.43	6 / 1	0.52	0.05	-90.38	4 / 0	0.34	0.00	-100
Bloomfield / Cathedral Oak	11.64/24.22	6 / 0	0.52	0.00	-100	4 / 0	0.34	0.00	-100
Bloomfield / Bit	11.64/20.86	6 / 0	0.52	0.00	-100	4 / 0	0.34	0.00	-100
Bloomfield / Molera	11.64/31.90	6 / 0	0.52	0.00	-100	4 / 0	0.34	0.00	-100
Bloomfield / Moss Landing	11.64/33.54	6 / 1	0.52	0.03	-94.23	4 / 0	0.34	0.00	-100
Bloomfield / Jensen	11.64/33.36	6 / 4	0.52	0.12	-76.92	4 / 2	0.34	0.06	-82.35
Bloomfield / Meridian	11.64/24.22	6 / 3	0.52	0.12	-76.92	4 / 2	0.34	0.08	-76.47

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.9: Comparison of Accident Rates for Two-Lane Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w)(a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Watsonville / SR 183 & I	2.68/15.57	2 / 2	0.75	0.13	-82.67	2 / 2	0.75	0.13	-82.67
Watsonville / Uvas	2.68/3.08	2 / 0	0.75	0.00	-100	2 / 0	0.75	0.00	-100
Watsonville / Los Carneros	2.68/10.13	2 / 0	0.75	0.00	-100	2 / 0	0.75	0.00	-100
Watsonville / Cuttings Wharf	2.68/10.95	2 / 0	0.75	0.00	-100	2 / 0	0.75	0.00	-100
Watsonville / Nicolaus	2.68/4.29	2 / 0	0.75	0.00	-100	2 / 0	0.75	0.00	-100
Watsonville / Madison	2.68/8.60	2 / 1	0.75	0.12	-84	2 / 0	0.75	0.00	-100
Left Turn									
Watsonville / Salinas	5.37/31.17	1 / 15	0.19	0.48	152.63	1 / 8	0.19	0.26	36.84
Watsonville / Torero	5.37/21.43	1 / 1	0.19	0.05	-73.68	1 /	0.19	0.00	-100
Watsonville / Cathedral Oak	5.37/24.22	1 / 0	0.19	0.00	-100	1 /	0.19	0.00	-100
Watsonville / Bit	5.37/20.86	1 / 0	0.19	0.00	-100	1 /	0.19	0.00	-100
Watsonville / Molera	5.37/31.90	1 / 0	0.19	0.00	-100	1 /	0.19	0.00	-100
Watsonville / Moss Landing	5.37/33.54	1 / 1	0.19	0.03	-84.21	1 /	0.19	0.00	-100
Watsonville / Jensen	5.37/33.36	1 / 4	0.19	0.12	-36.84	1 / 2	0.19	0.06	-68.42
Watsonville / Meridian	5.37/24.22	1 / 3	0.19	0.12	-36.84	1 / 2	0.19	0.08	-57.89

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.10: Comparison of Accident Rates for Two-Lane Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Espinosa / SR 183 & 1	8.38/15.57	3 / 2	0.36	0.13	-63.89	0 / 2	0.00	0.12	
Espinosa / Uvas	8.38/3.08	3 / 0	0.36	0.00	-100	0 / 0	0.00	0.00	
Espinosa / Los Cameros	8.38/10.13	3 / 0	0.36	0.00	-100	0 / 0	0.00	0.00	
Espinosa / Cuttings Wharf	8.38/10.95	3 / 0	0.36	0.00	-100	0 / 0	0.00	0.00	
Espinosa / Nicolaus	8.38/4.29	3 / 0	0.36	0.00	-100	0 / 0	0.00	0.00	
Espinosa / Madison	8.38/8.60	3 / 1	0.36	0.12	-66.67	0 / 0	0.00	0.00	
Left Turn									
Espinosa / Salinas	16.75/31.17	0 / 15	0.00	0.48		0 / 8	0.00	0.26	
Espinosa / Torero	16.75/21.43	0 / 1	0.00	0.05		0 / 0	0.00	0.00	
Espinosa / Cathedral Oak	16.75/24.22	0 / 0	0.00	0.00		0 / 0	0.00	0.00	
Espinosa / Bit	16.75/20.86	0 / 0	0.00	0.00		0 / 0	0.00	0.00	
Espinosa / Molera	16.75/31.90	0 / 0	0.00	0.00		0 / 0	0.00	0.00	
Espinosa / Moss Landing	16.75/33.54	0 / 1	0.00	0.03		0 / 0	0.00	0.00	
Espinosa / Jensen	16.75/33.36	0 / 4	0.00	0.12		0 / 2	0.00	0.06	
Espinosa / Meridian	16.75/24.22	0 / 3	0.00	0.12		0 / 2	0.00	0.08	

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.11: Comparison of Accident Rates for Two-Lane Highways - Avoidable Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (s)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Fairview / SR 183 & 1	4.49/15.57	1/2	0.11	0.13	18.18	1/2	0.11	0.13	18.18
Fairview / Uvas	4.49/3.08	1/0	0.11	0.00	-100	1/0	0.11	0.00	-100
Fairview / Los Cameros	4.49/10.13	1/0	0.11	0.00	-100	1/0	0.11	0.00	-100
Fairview / Cuttings Wharf	4.49/10.95	1/0	0.11	0.00	-100	1/0	0.11	0.00	-100
Fairview / Nicolaus	4.49/4.29	1/0	0.11	0.00	-100	1/0	0.11	0.00	-100
Fairview / Madison	4.49/8.60	1/1	0.11	0.12	9.09	1/0	0.11	0.00	-100
Left Turn									
Fairview / Salinas	8.98/31.17	2/15	0.11	0.48	336.36	1/8	0.06	0.26	333.33
Fairview / Torero	8.98/21.43	2/1	0.11	0.05	-54.55	1/0	0.06	0.00	-100
Fairview / Cathedral Oak	8.98/24.22	2/0	0.11	0.00	-100	1/0	0.06	0.00	-100
Fairview / Bit	8.98/20.86	2/0	0.11	0.00	-100	1/0	0.06	0.00	-100
Fairview / Molera	8.98/31.90	2/0	0.11	0.00	-100	1/0	0.06	0.00	-100
Fairview / Moss Landing	8.98/33.54	2/1	0.11	0.03	-72.73	1/0	0.06	0.00	-100
Fairview / Jensen	8.98/33.36	2/4	0.11	0.12	9.09	1/2	0.06	0.06	0
Fairview / Meridian	8.98/24.22	2/3	0.11	0.12	9.09	1/2	0.06	0.08	33.33

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.12: Comparison of Accident Rates for Four-Lane Wide Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Spence / Elverta	13.87/12.05	10 / 7	0.72	0.58	-19.44	8 / 6	0.58	0.50	-13.79
Spence / Catlett	13.87/10.11	7 / 0	0.51	0.00	-100	5 / 0	0.36	0.00	-100
Spence / Espinosa	13.87/24.22	7 / 5	0.51	0.21	-58.82	5 / 3	0.36	0.12	-66.67
Spence / Castro Valley	13.87/24.09	6 / 6	0.43	0.25	-41.86	4 / 4	0.29	0.17	-41.38
Spence / Ocean	13.87/29.84	5 / 7	0.36	0.23	-36.11	3 / 6	0.22	0.20	-9.09
Spence / Bay Front	13.87/17.06	5 / 3	0.36	0.18	-50	3 / 1	0.22	0.06	-72.73
Left Turn									
Spence / Elverta	27.74/24.09	10 / 7	0.36	0.29	-19.44	4 / 3	0.14	0.12	-14.29
Spence / Catlett	27.74/19.49	10 / 3	0.36	0.15	-58.33	4 / 2	0.14	0.10	-28.57
Spence / Bell Cr.	27.74/18.82	8 / 1	0.29	0.05	-82.76	3 / 0	0.11	0.00	-100
Spence / Tower	27.74/37.16	8 / 8	0.29	0.22	-24.14	3 / 3	0.11	0.08	-27.27
Spence / Solano	27.74/20.88	8 / 4	0.29	0.19	-34.48	3 / 3	0.11	0.14	27.27
Spence / Oak Knoll	27.74/21.48	8 / 17	0.29	0.79	172.41	3 / 11	0.11	0.51	363.64

(a) For right turn acceleration lane the volume is for one direction only.

TABLE 5.13: Comparison of Accident Rates for Four-Lane Wide Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
McCloskey / Elverta	7.37/12.05	19 / 7	2.58	0.58	-77.52	10 / 6	1.36	0.50	-63.24
McCloskey / Catlett	7.37/10.11	18 / 0	2.44	0.00	-100	10 / 0	1.36	0.00	-100
McCloskey / Espinosa	7.37/24.22	18 / 5	2.44	0.21	-91.39	10 / 3	1.36	0.12	-91.18
McCloskey / Castro Valley	7.37/24.09	17 / 6	2.31	0.25	-89.18	10 / 4	1.36	0.17	-87.50
McCloskey / Ocean	7.37/29.84	17 / 7	2.31	0.23	-90.04	10 / 6	1.36	0.20	-85.29
McCloskey / Bay Front	7.37/17.06	17 / 3	2.31	0.18	-92.21	10 / 1	1.36	0.06	-95.59
Left Turn									
McCloskey / Elverta	14.75/24.09	20 / 7	1.36	0.29	-78.68	12 / 3	0.81	0.12	-85.19
McCloskey / Catlett	14.75/19.49	20 / 3	1.36	0.15	-88.97	12 / 2	0.81	0.10	-87.65
McCloskey / Bell Cr.	14.75/18.82	17 / 1	1.15	0.05	-95.65	10 / 0	0.68	0.00	-100
McCloskey / Tower	14.75/37.16	17 / 8	1.15	0.22	-80.87	10 / 3	0.68	0.08	-88.24
McCloskey / Solano	14.75/20.88	17 / 4	1.15	0.19	-83.48	10 / 3	0.68	0.14	-79.41
McCloskey / Oak Knoll	14.75/21.48	17 / 17	1.15	0.79	-31.30	10 / 11	0.68	0.51	-25

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.14: Comparison of Accident Rates for Four-Lane Wide Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Yerba Buena / Elverta	8.60/12.05	7/7	0.81	0.58	-28.40	1/6	0.12	0.50	316.67
Yerba Buena / Callett	8.60/10.11	7/0	0.81	0.00	-100	1/0	0.12	0.00	-100
Yerba Buena / Espinosa	8.60/24.22	7/5	0.81	0.21	-74.07	1/3	0.12	0.12	0
Yerba Buena / Castro Valley	8.60/24.09	7/6	0.81	0.25	-69.14	1/4	0.12	0.17	41.67
Yerba Buena / Ocean	8.60/29.84	7/7	0.81	0.23	-71.60	1/6	0.12	0.20	66.67
Yerba Buena / Bay Front	8.60/17.06	7/3	0.81	0.18	-77.78	1/1	0.12	0.06	-50
Left Turn									
Yerba Buena / Elverta	17.19/24.09	10/7	0.58	0.29	-50	2/3	0.12	0.12	0
Yerba Buena / Callett	17.19/19.49	10/3	0.58	0.15	-74.14	2/2	0.12	0.10	-16.67
Yerba Buena / Bell Cr.	17.19/18.82	10/1	0.58	0.05	-91.38	2/0	0.12	0.00	-100
Yerba Buena / Tower	17.19/37.16	10/8	0.58	0.22	-62.07	2/3	0.12	0.08	-33.33
Yerba Buena / Solano	17.19/20.88	9/4	0.52	0.19	-63.46	2/3	0.12	0.14	16.67
Yerba Buena / Oak Knoll	17.19/21.48	9/17	0.52	0.79	51.92	2/11	0.12	0.51	325

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.15: Comparison of Accident Rates for Four-Lane Narrow Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Tustin / Blackie	25.49/24.31	14 / 13	0.55	0.53	-3.64	8 / 6	0.31	0.25	-19.35
Tustin / Monte-Vina	25.49/36.46	12 / 9	0.47	0.25	-47.57	7 / 6	0.27	0.16	-40.74
Tustin / Pesante	25.49/24.31	12 / 17	0.47	0.70	48.94	7 / 12	0.27	0.49	81.48
Tustin / Summit - NB	25.49/32.07	12 / 14	0.47	0.44	-6.38	7 / 5	0.27	0.16	-40.74
Tustin / Bay Front	25.49/17.06	12 / 3	0.47	0.18	-61.70	7 / 1	0.27	0.06	-77.78
Tustin / Summit - SB	25.49/32.07	12 / 9	0.47	0.28	-40.43	7 / 4	0.27	0.12	-55.56
Left Turn									
Tustin / Black	50.97/72.45	10 / 10	0.20	0.14	-30	4 / 4	0.08	0.06	-25
Tustin / Dunbarlon	50.97/50.52	10 / 10	0.20	0.20	0	4 / 4	0.08	0.08	0
Tustin / Messick #1	50.97/50.97	10 / 8	0.20	0.16	-20	4 / 2	0.08	0.04	-50
Tustin / Glenwood	50.97/58.25	9 / 17	0.18	0.29	64.24	4 / 7	0.08	0.12	50
Tustin / Sugar Loaf	50.97/58.25	9 / 7	0.18	0.12	-33.33	4 / 3	0.08	0.05	-37.50
Tustin / Glenwood Cutoff	50.97/58.25	9 / 4	0.18	0.07	-61.11	4 / 2	0.08	0.03	-62.50

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.16: Comparison of Accident Rates for Four-Lane Narrow Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Echo Valley / Blackie	25.49/24.31	16 / 13	0.63	0.53	-15.87	8 / 6	0.31	0.25	-19.35
Echo Valley / Monte-Vina	25.49/36.46	14 / 9	0.55	0.25	-55.12	8 / 6	0.31	0.16	-48.39
Echo Valley / Pesante	25.49/24.31	14 / 17	0.55	0.70	27.27	8 / 12	0.31	0.49	58.06
Echo Valley / Summit - NB	25.49/32.07	12 / 14	0.47	0.44	-6.54	6 / 5	0.24	0.16	-32.03
Echo Valley / Bay Front	25.49/17.06	14 / 3	0.55	0.18	-67.27	8 / 1	0.31	0.06	-80.65
Echo Valley / Summit - SB	25.49/32.07	14 / 9	0.55	0.28	-49.09	8 / 4	0.31	0.12	-61.29
Left Turn									
Echo Valley / Black	50.97/72.45	10 / 10	0.20	0.14	-30	6 / 4	0.12	0.06	-50
Echo Valley / Dunbarton	50.97/50.52	10 / 10	0.20	0.20	0	6 / 4	0.12	0.08	-33.33
Echo Valley / Messick #1	50.97/50.97	9 / 8	0.18	0.16	-11.11	5 / 2	0.10	0.04	-60
Echo Valley / Glenwood	50.97/58.25	9 / 17	0.18	0.29	61.11	5 / 7	0.10	0.12	20
Echo Valley / Sugar Loaf	50.97/58.25	9 / 7	0.18	0.12	-33.33	5 / 3	0.10	0.05	-50
Echo Valley / Glenwood C.	50.97/58.25	9 / 4	0.18	0.07	-61.11	5 / 2	0.10	0.03	-70

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.17: Comparison of Accident Rates for Four-Lane Narrow Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Crazy Horse / Blackie	25.49/24.31	14 / 13	0.55	0.53	-3.64	7 / 6	0.27	0.25	-7.41
Crazy Horse / Monte-Vina	25.49/36.46	14 / 9	0.55	0.25	-55.12	7 / 6	0.27	0.16	-40.74
Crazy Horse / Pesante	25.49/24.31	14 / 17	0.55	0.70	27.27	7 / 12	0.27	0.49	81.48
Crazy Horse / Summit - NB	25.49/32.07	12 / 14	0.47	0.44	-6.38	6 / 5	0.24	0.16	-33.33
Crazy Horse / Bay Front	25.49/17.06	12 / 3	0.47	0.18	-61.70	6 / 1	0.24	0.06	-75
Crazy Horse / Summit - SB	25.49/32.07	12 / 9	0.47	0.28	-40.43	6 / 4	0.24	0.12	-50
Left Turn									
Crazy Horse / Black	50.97/72.45	26 / 10	0.51	0.14	-72.55	10 / 4	0.20	0.06	-70
Crazy Horse / Dunbarton	50.97/50.52	26 / 10	0.51	0.20	-60.78	10 / 4	0.20	0.08	-60
Crazy Horse / Messick #1	50.97/50.97	26 / 8	0.51	0.16	-68.63	10 / 2	0.20	0.04	-80
Crazy Horse / Glenwood	50.97/58.25	26 / 17	0.51	0.29	-43.14	10 / 7	0.20	0.12	-40
Crazy Horse / Sugar Loaf	50.97/58.25	25 / 7	0.49	0.12	-75.51	10 / 3	0.20	0.05	-75
Crazy Horse / Glenwood C.	50.97/58.25	25 / 4	0.49	0.07	-85.71	10 / 2	0.20	0.03	-85

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.18: Comparison of Accident Rates for Two-Lane Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Bloomfield / SR 183 & I	5.82/15.57	9 / 19	1.55	1.22	-21.29	6 / 9	1.03	0.58	-43.69
Bloomfield / Uvas	5.82/3.08	9 / 0	1.55	0.00	-100.00	6 / 0	1.03	0.00	-100.00
Bloomfield / Los Cameros	5.82/10.13	8 / 0	1.37	0.00	-100.00	5 / 0	0.86	0.00	-100.00
Bloomfield / Cuttings Wharf	5.82/10.95	7 / 1	1.20	0.09	-92.50	4 / 1	0.69	0.09	-86.96
Bloomfield / Nicolaus	5.82/4.29	7 / 0	1.20	0.00	-100.00	4 / 0	0.69	0.00	-100.00
Bloomfield / Madison	5.82/8.60	7 / 9	1.20	1.05	-12.50	4 / 5	0.69	0.58	-15.94
Left Turn									
Bloomfield / Salinas	11.64/31.17	13 / 35	1.12	1.12	0.00	6 / 12	0.52	0.39	-25.00
Bloomfield / Torero	11.64/21.43	12 / 6	1.03	0.28	-72.82	6 / 3	0.52	0.14	-73.08
Bloomfield / Cathedral Oak	11.64/24.22	12 / 2	1.03	0.08	-92.23	6 / 0	0.52	0.00	-100.00
Bloomfield / Bit	11.64/20.86	11 / 3	0.95	0.14	-85.26	5 / 3	0.43	0.14	-67.44
Bloomfield / Molera	11.64/31.90	11 / 4	0.95	0.13	-86.32	5 / 3	0.43	0.09	-79.07
Bloomfield / Moss Landing	11.64/33.54	11 / 1	0.95	0.03	-96.84	5 / 0	0.43	0.00	-100.00
Bloomfield / Jensen	11.64/33.36	11 / 8	0.95	0.24	-74.74	5 / 3	0.43	0.09	-79.07
Bloomfield / Meridian	11.64/24.22	11 / 6	0.95	0.25	-73.68	5 / 2	0.43	0.08	-81.40

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.19: Comparison of Accident Rates for Two-Lane Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Watsonville / SR 183 & 1	2.68/15.57	6 / 19	2.24	1.22	-45.54	4 / 9	1.49	0.58	-61.07
Watsonville / Uvas	2.68/3.08	6 / 0	2.24	0.00	-100.00	4 / 0	1.49	0.00	-100.00
Watsonville / Los Cameros	2.68/10.13	6 / 0	2.24	0.00	-100.00	4 / 0	1.49	0.00	-100.00
Watsonville / Cuttings Wharf	2.68/10.95	6 / 1	2.24	0.09	-95.98	4 / 1	1.49	0.09	-93.96
Watsonville / Nicolaus	2.68/4.29	6 / 0	2.24	0.00	-100.00	4 / 0	1.49	0.00	-100.00
Watsonville / Madison	2.68/8.60	6 / 9	2.24	1.05	-53.13	4 / 5	1.49	0.58	-61.07
Left Turn									
Watsonville / Salinas	5.37/31.17	7 / 35	1.30	1.12	-13.85	5 / 12	0.93	0.39	-58.06
Watsonville / Totoro	5.37/21.43	7 / 6	1.30	0.28	-78.46	5 / 3	0.93	0.14	-84.95
Watsonville / Cathedral Oak	5.37/24.22	7 / 2	1.30	0.08	-93.85	5 / 0	0.93	0.00	-100.00
Watsonville / Bit	5.37/20.86	7 / 3	1.30	0.14	-89.23	5 / 2	0.93	0.14	-84.95
Watsonville / Molera	5.37/31.90	7 / 4	1.30	0.13	-90.00	5 / 3	0.93	0.09	-90.32
Watsonville / Moss Landing	5.37/33.54	7 / 1	1.30	0.03	-97.69	5 / 0	0.93	0.00	-100.00
Watsonville / Jensen	5.37/33.36	7 / 8	1.30	0.24	-81.54	5 / 3	0.93	0.09	-90.32
Watsonville / Meridian	5.37/24.22	7 / 6	1.30	0.25	-80.77	5 / 2	0.93	0.08	-91.40

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.20: Comparison of Accident Rates for Two-Lane Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Espinosa / SR 183 & 1	8.38/15.57	7 / 19	0.84	1.22	45.24	1 / 9	0.12	0.58	383.33
Espinosa / Uvas	8.38/3.08	7 / 0	0.84	0.00	-100.00	1 / 0	0.12	0.00	-100.00
Espinosa / Los Cameros	8.38/10.13	7 / 0	0.84	0.00	-100.00	1 / 0	0.12	0.00	-100.00
Espinosa / Cuttings Wharf	8.38/10.95	7 / 1	0.84	0.09	-89.29	1 / 1	0.12	0.09	-25.00
Espinosa / Nicolaus	8.38/4.29	7 / 0	0.84	0.00	-100.00	1 / 0	0.12	0.00	-100.00
Espinosa / Madison	8.38/8.60	7 / 9	0.84	1.05	25.00	1 / 5	0.12	0.58	383.33
Left Turn									
Espinosa / Salinas	16.75/31.17	8 / 35	0.48	1.12	133.33	1 / 12	0.06	0.39	550.00
Espinosa / Torero	16.75/21.43	8 / 6	0.48	0.28	-41.67	1 / 3	0.06	0.14	133.33
Espinosa / Cathedral Oak	16.75/24.22	8 / 2	0.48	0.08	-83.33	1 / 3	0.06	0.00	-100.00
Espinosa / Bit	16.75/20.86	8 / 3	0.48	0.14	-70.83	1 / 3	0.06	0.14	133.33
Espinosa / Molera	16.75/31.90	8 / 4	0.48	0.13	-72.92	1 / 3	0.06	0.09	50.00
Espinosa / Moss Landing	16.75/33.54	8 / 1	0.48	0.03	-93.75	1 / 0	0.06	0.00	-100.00
Espinosa / Jensen	16.75/33.36	8 / 8	0.48	0.24	-50.00	1 / 3	0.06	0.09	50.00
Espinosa / Meridian	16.75/24.22	8 / 6	0.48	0.25	-47.92	1 / 2	0.06	0.08	33.33

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.21: Comparison of Accident Rates for Two-Lane Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)	Injury & Fatal Acc. (w/o vs. w)	Accident Rate w/o Accel. Lane	Accident Rate with Accel. Lane	Difference (%)
Right Turn									
Fairview / SR 183 & 1	4.49/15.57	10 / 19	2.23	1.22	-45.29	4 / 9	0.89	0.58	-34.83
Fairview / Uvas	4.49/3.08	10 / 0	2.23	0.00	-100.00	4 / 0	0.89	0.00	-100.00
Fairview / Los Cameros	4.49/10.13	10 / 0	2.23	0.00	-100.00	4 / 0	0.89	0.00	-100.00
Fairview / Cuttings Wharf	4.49/10.95	10 / 1	2.23	0.09	-95.96	4 / 1	0.89	0.09	-89.89
Fairview / Nicolaus	4.49/4.29	10 / 0	2.23	0.00	-100.00	4 / 0	0.89	0.00	-100.00
Fairview / Madison	4.49/8.60	10 / 9	2.23	1.05	-52.91	4 / 5	0.89	0.58	-34.83
Left Turn									
Fairview / Salinas	8.98/31.17	21 / 35	2.34	1.12	-52.14	11 / 12	1.22	0.39	-68.03
Fairview / Torero	8.98/21.43	21 / 6	2.34	0.28	-88.03	11 / 3	1.22	0.14	-88.52
Fairview / Cathedral Oak	8.98/24.22	21 / 2	2.34	0.08	-96.58	11 / 0	1.22	0.00	-100.00
Fairview / Bit	8.98/20.86	21 / 3	2.34	0.14	-94.02	11 / 3	1.22	0.14	-88.52
Fairview / Molera	8.98/31.90	21 / 4	2.34	0.13	-94.44	11 / 3	1.22	0.09	-92.62
Fairview / Moss Landing	8.98/33.54	21 / 1	2.34	0.03	-98.72	11 / 0	1.22	0.00	-100.00
Fairview / Jensen	8.98/33.36	21 / 8	2.34	0.24	-89.74	11 / 3	1.22	0.09	-92.62
Fairview / Meridian	8.98/24.22	21 / 6	2.34	0.25	-89.32	11 / 2	1.22	0.08	-93.44

(a) For Right Turn acceleration lanes the volume is for one direction only.

TABLE 5.22: Summary of Average Accident Rates According to Sites Without Acceleration Lane - All Accidents

Control Sites vs. Sites w/ Accel. Lane (w/o vs. w)	Total Acc. Rate of All Sites w/o Accel. Lane	Total Acc. Rate of All Sites w/ Accel. Lane	Difference (%)	Injury Acc. Rate of All Sites w/o Accel. Lane	Injury Acc. Rate of All Sites w/ Accel. Lane	Difference (%)
Right Turn:						
4-Lane-Wide Median						
Spence / All Sites w/ Accel. Lane	0.48	0.24	-50.37	0.34	0.17	-49.35
McCloskey / All Sites w/ Accel. Lane	2.40	0.24	-90.05	1.36	0.17	-87.44
Yerba Buena / All Sites w/ Accel. Lane	0.81	0.24	-70.69	0.12	0.17	46.55
Average	1.01	0.22	-78.06	0.49	0.15	-70.25
4-Lane-Narrow Median						
Tustin / All Sites w/ Accel. Lane	0.48	0.39	-19.21	0.28	0.20	-27.51
Echo Valley / All Sites w/ Accel. Lane	0.55	0.39	-28.83	0.30	0.20	-32.24
Crazy Horse / All Sites w/ Accel. Lane	0.50	0.39	-21.34	0.26	0.20	-20.08
Average	0.47	0.41	-13.43	0.27	0.21	-23.16
2-Lane						
Bloomfield / All Sites w/ Accel. Lane	1.35	0.55	-59.05	0.83	0.29	-65.67
Watsonville / All Sites w/ Accel. Lane	2.24	0.55	-75.38	1.49	0.29	-80.90
Espinoza / All Sites w/ Accel. Lane	0.84	0.55	-34.02	0.12	0.29	138.88
Fairview / All Sites w/ Accel. Lane	2.23	0.55	-75.25	0.89	0.29	-68.00
Average	1.44	0.55	-61.80	0.65	0.29	-55.96
Average for Right Turns	0.81	0.38	-53.28	0.41	0.20	-50.36
Left Turn:						
4-Lane-Wide Median						
Spence / All Sites w/ Accel. Lane	0.31	0.28	-9.79	0.12	0.16	29.01
McCloskey / All Sites w/ Accel. Lane	1.22	0.28	-76.90	0.72	0.16	-78.56
Yerba Buena / All Sites w/ Accel. Lane	0.56	0.28	-49.88	0.12	0.16	33.24
Average	0.61	0.31	-49.88	0.26	0.17	-34.68
4-Lane-Narrow Median						
Tustin / All Sites w/ Accel. Lane	0.19	0.16	-13.83	0.08	0.06	-19.60
Echo Valley / All Sites w/ Accel. Lane	0.18	0.16	-12.29	0.10	0.06	-39.70
Crazy Horse / All Sites w/ Accel. Lane	0.50	0.16	-68.11	0.20	0.06	-67.84
Average	0.29	0.16	-44.81	0.13	0.06	-50.10
2-Lane						
Bloomfield / All Sites w/ Accel. Lane	0.99	0.29	-70.19	0.57	0.12	-79.30
Watsonville / All Sites w/ Accel. Lane	1.30	0.29	-77.41	0.93	0.12	-87.35
Espinoza / All Sites w/ Accel. Lane	0.48	0.29	-38.34	0.06	0.12	97.30
Fairview / All Sites w/ Accel. Lane	2.34	0.29	-87.41	1.22	0.12	-90.38
Average	1.11	0.29	-73.50	0.52	0.12	-77.50
Average for Left Turns	0.49	0.20	-59.01	0.24	0.10	-58.40

TABLE 5.23: Summary of Average Accident Rates According to Sites With Right Turn Acceleration Lane - All Accidents

Control Sites vs. Sites w/ Accel. Lane (w/o vs. w)	Total Acc. Rate of All Sites w/o Accel. Lane	Total Acc. Rate of All Sites w/ Accel. Lane	Difference (%)	Injury Acc. Rate of All Sites w/o Accel. Lane	Injury Acc. Rate of All Sites w/ Accel. Lane	Difference (%)
4-Lane-Wide Median						
All Control Sites / Espinosa	1.07	0.21	-80.42	0.54	0.12	-77.65
All Control Sites / Castro Valley	1.01	0.25	-75.13	0.50	0.17	-66.23
All Control Sites / Ocean	0.97	0.23	-76.33	0.47	0.20	-57.43
All Control Sites / Bay Front	0.97	0.18	-81.48	0.47	0.06	-87.23
Average	0.75	0.22	-70.75	0.49	0.15	-70.25
4-Lane-Narrow Median						
All Control Sites / Pesante	0.52	0.70	33.82	0.29	0.49	70.32
All Control Sites / Summit - NB	0.47	0.44	-6.54	0.25	0.16	-35.60
All Control Sites / Bay Front	0.50	0.18	-63.78	0.27	0.06	-78.15
All Control Sites / Summit - SB	0.50	0.28	-43.65	0.27	0.12	-56.30
Average	0.50	0.41	-17.99	0.27	0.21	-23.16
2-Lane						
All Control Sites / SR 183	1.50	1.22	-18.53	0.70	0.58	-17.37
All Control Sites / Uvas	1.50	0.00	-100.00	0.70	0.09	-87.18
All Control Sites / Los Cameros	1.45	0.00	-100.00	0.66	0.00	-100.00
All Control Sites / Cuttings Wharf	1.40	0.09	-93.59	0.61	0.09	-85.21
All Control Sites / Nicolaus	1.40	0.00	-100.00	0.61	0.00	-100.00
All Control Sites / Madison	1.40	1.05	-25.21	0.61	0.58	-4.66
Average	1.44	0.55	-61.80	0.65	0.29	-55.96
Average for Right Turns	0.77	0.37	-52.42	0.41	0.20	-50.48

TABLE 5.24: Summary of Average Accident Rates According to Sites With Left Turn Acceleration Lane - All Accidents

Control Sites vs. Sites w/ Accel. Lane (w/o vs. w)	Total Acc. Rate of All Sites w/o Accel. Lane	Total Acc. Rate of All Sites w/ Accel. Lane	Difference (%)	Injury Acc. Rate of All Sites w/o Accel. Lane	Injury Acc. Rate of All Sites w/ Accel. Lane	Difference (%)
4-Lane-Wide Median						
All Control Sites / Bell Creek	0.59	0.05	-91.47	0.25	0.00	-100.00
All Control Sites / Tower	0.59	0.22	-62.49	0.25	0.08	-68.17
All Control Sites / Solano	0.57	0.19	-66.65	0.25	0.14	-44.30
All Control Sites / Oak Knoll	0.57	0.79	38.67	0.25	0.51	102.91
Average	0.58	0.31	-47.23	0.25	0.17	-31.22
4-Lane-Narrow Median						
All Control Sites / Black	0.30	0.14	-53.46	0.13	0.06	-54.13
All Control Sites / Dunbarton	0.30	0.20	-33.52	0.13	0.08	-38.84
All Control Sites / Messick #1	0.29	0.16	-45.63	0.12	0.04	-67.81
All Control Sites / Glenwood	0.29	0.29	0.00	0.12	0.12	-3.43
All Control Sites / Sugar Loaf	0.28	0.12	-57.33	0.12	0.05	-59.76
All Control Sites / Glenwood Cutoff	0.28	0.07	-75.11	0.12	0.03	-75.86
Average	0.29	0.16	-44.81	0.13	0.06	-50.10
2-Lane						
All Control Sites / Salinas	1.15	1.12	-2.31	0.51	0.39	-24.23
All Control Sites / Torero	1.12	0.28	-75.07	0.51	0.14	-72.80
All Control Sites / Cathedral Oak	1.12	0.08	-92.88	0.51	0.00	-100.00
All Control Sites / Bit	1.10	0.14	-87.27	0.49	0.14	-71.51
All Control Sites / Molera	1.10	0.12	-89.09	0.49	0.09	-81.68
All Control Sites / Moss Landing	1.10	0.03	-97.27	0.49	0.00	-100.00
All Control Sites / Jensen	1.10	0.24	-78.18	0.49	0.09	-81.68
All Control Sites / Meridian	1.10	0.25	-77.27	0.49	0.08	-83.72
Average	1.11	0.29	-73.50	0.52	0.12	-77.50
Average for Left Turns	0.52	0.23	-56.29	0.24	0.10	-58.92

TABLE 5.25: Comparison of Accident Cost for Four-Lane Wide Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Spence / Elverta	13.87/12.05	10 / 7	8 / 6	2 / 1	\$162,400 / \$120,200	\$11,709	\$9,975	-14.81
Spence / Catlett	13.87/10.11	7 / 0	5 / 0	2 / 0	\$103,900 / \$0	\$7,491	\$0	-100
Spence / Espinosa	13.87/24.22	7 / 5	5 / 3	2 / 2	\$103,900 / \$64,900	\$7,491	\$2,680	-64.22
Spence / Castro Valley	13.87/24.09	6 / 6	4 / 4	2 / 2	\$84,400 / \$84,400	\$6,085	\$3,504	-42.42
Spence / Ocean	13.87/29.84	5 / 7	3 / 6	2 / 1	\$64,900 / \$120,200	\$4,679	\$4,028	-13.91
Spence / Bay Front	13.87/17.06	5 / 3	3 / 1	2 / 2	\$64,900 / \$25,900	\$4,679	\$1,518	-67.56
Left Turn								
Spence / Elverta	27.74/24.09	10 / 7	4 / 3	6 / 4	\$97,200 / \$71,300	\$3,504	\$2,960	-15.53
Spence / Catlett	27.74/19.49	10 / 3	4 / 2	6 / 1	\$97,200 / \$42,200	\$3,504	\$2,165	-38.21
Spence / Bell Cr.	27.74/18.82	8 / 1	3 / 0	5 / 1	\$74,500 / \$3,200	\$2,686	\$170	-93.67
Spence / Tower	27.74/37.16	8 / 8	3 / 3	5 / 5	\$74,500 / \$74,500	\$2,686	\$2,005	-25.35
Spence / Solano	27.74/20.88	8 / 4	3 / 3	5 / 1	\$74,500 / \$61,700	\$2,686	\$2,955	10.01
Spence / Oak Knoll	27.74/21.48	8 / 17	3 / 11	5 / 6	\$74,500 / \$233,700	\$2,686	\$10,880	305.06

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.26: Comparison of Accident Cost for Four-Lane Wide Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
McCloskey / Elverta	7.37/12.05	19 / 7	10 / 6	9 / 1	\$223,800 / \$120,200	\$30,366	\$9,975	-67.15
McCloskey / Catlett	7.37/10.11	18 / 0	10 / 0	8 / 0	\$220,600 / \$0	\$29,932	\$0	-100
McCloskey / Espinosa	7.37/24.22	18 / 5	10 / 3	8 / 2	\$220,600 / \$64,900	\$29,932	\$2,680	-91.05
McCloskey / Castro Valley	7.37/24.09	17 / 6	10 / 4	7 / 2	\$217,400 / \$84,400	\$29,498	\$3,504	-88.12
McCloskey / Ocean	7.37/29.84	17 / 7	10 / 6	7 / 1	\$217,400 / \$120,200	\$29,498	\$4,028	-86.34
McCloskey / Bay Front	7.37/17.06	17 / 3	10 / 1	7 / 2	\$217,400 / \$25,900	\$29,498	\$1,518	-94.85
Left Turn								
McCloskey / Elverta	14.75/24.09	20 / 7	12 / 3	8 / 4	\$259,600 / \$71,300	\$17,600	\$2,960	-83.18
McCloskey / Catlett	14.75/19.49	20 / 3	12 / 2	8 / 1	\$259,600 / \$42,200	\$17,600	\$2,165	-87.70
McCloskey / Bell Cr.	14.75/18.82	17 / 1	10 / 0	7 / 1	\$217,400 / \$3,200	\$14,739	\$170	-98.85
McCloskey / Tower	14.75/37.16	17 / 8	10 / 3	7 / 5	\$217,400 / \$74,500	\$14,739	\$2,005	-86.40
McCloskey / Solano	14.75/20.88	17 / 4	10 / 3	7 / 1	\$217,400 / \$61,700	\$14,739	\$2,955	-79.95
McCloskey / Oak Knoll	14.75/21.48	17 / 17	10 / 11	7 / 6	\$217,400 / \$233,700	\$14,739	\$10,880	-26.18

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.27: Comparison of Accident Cost for Four-Lane Wide Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Yerba Buena / Elverta	8.60/12.05	7/7	1/6	6/1	\$38,700 / \$120,200	\$4,500	\$9,975	121.67
Yerba Buena / Catlett	8.60/10.11	7/0	1/0	6/0	\$38,700 / \$0	\$4,500	\$0	-100
Yerba Buena / Espinosa	8.60/24.22	7/5	1/3	6/2	\$38,700 / \$64,900	\$4,500	\$2,680	-40.44
Yerba Buena / Castro Valley	8.60/24.09	7/6	1/4	6/2	\$38,700 / \$84,400	\$4,500	\$3,504	-22.13
Yerba Buena / Ocean	8.60/29.84	7/7	1/6	6/1	\$38,700 / \$120,200	\$4,500	\$4,028	-10.49
Yerba Buena / Bay Front	8.60/17.06	7/3	1/1	6/2	\$38,700 / \$25,900	\$4,500	\$1,518	-66.27
Left Turn								
Yerba Buena / Elverta	17.19/24.09	10/7	2/3	8/4	\$64,600 / \$71,300	\$3,758	\$2,960	-21.23
Yerba Buena / Catlett	17.19/19.49	10/3	2/2	8/1	\$64,600 / \$42,200	\$3,758	\$2,165	-42.39
Yerba Buena / Bell Cr.	17.19/18.82	10/1	2/0	8/1	\$64,600 / \$3,200	\$3,758	\$170	-95.48
Yerba Buena / Tower	17.19/37.16	10/8	2/3	8/5	\$64,600 / \$74,500	\$3,758	\$2,005	-46.65
Yerba Buena / Solano	17.19/20.88	9/4	2/3	8/1	\$61,400 / \$61,700	\$3,572	\$2,955	-17.27
Yerba Buena / Oak Knoll	17.19/21.48	9/17	2/11	8/6	\$61,400 / \$233,700	\$3,572	\$10,880	204.59

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.28: Comparison of Accident Cost for Four-Lane Narrow Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Tustin / Blackie	25.49/24.31	14 / 13	8 / 6	6 / 7	\$175,200 / \$139,400	\$6,873	\$5,734	-16.58
Tustin / Monte-Vina	25.49/36.46	12 / 9	7 / 6	5 / 3	\$152,500 / \$126,600	\$5,983	\$3,472	-41.96
Tustin / Pesante	25.49/24.31	12 / 17	7 / 12	5 / 5	\$152,500 / \$250,000	\$5,983	\$10,284	71.89
Tustin / Summit - NB	25.49/32.07	12 / 14	7 / 5	5 / 9	\$152,500 / \$126,300	\$5,983	\$3,938	-34.18
Tustin / Bay Front	25.49/17.06	12 / 3	7 / 1	5 / 2	\$152,500 / \$25,900	\$5,983	\$1,518	-74.63
Tustin / Summit - SB	25.49/32.07	12 / 9	7 / 4	5 / 5	\$152,500 / \$94,000	\$5,983	\$2,931	-51.01
Left Turn								
Tustin / Black	50.97/72.45	10 / 10	4 / 4	6 / 6	\$97,200 / \$97,200	\$1,907	\$1,342	-29.63
Tustin / Dunbarton	50.97/50.52	10 / 10	4 / 4	6 / 6	\$97,200 / \$97,200	\$1,907	\$1,924	0.89
Tustin / Messick #1	50.97/50.97	10 / 8	4 / 2	6 / 6	\$97,200 / \$58,200	\$1,907	\$1,142	-40.12
Tustin / Glenwood	50.97/58.25	9 / 17	4 / 7	5 / 10	\$94,000 / \$168,500	\$1,844	\$2,893	56.87
Tustin / Sugar Loaf	50.97/58.25	9 / 7	4 / 3	5 / 4	\$94,000 / \$71,300	\$1,844	\$1,224	-33.63
Tustin / Glenwood Cutoff	50.97/58.25	9 / 4	4 / 2	5 / 2	\$94,000 / \$45,400	\$1,844	\$779	-57.74

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.29: Comparison of Accident Cost for Four-Lane Narrow Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Echo Valley / Blackie	25.49/24.31	16 / 13	8 / 6	8 / 7	\$181,600 / \$139,400	\$7,124	\$5,734	-19.52
Echo Valley / Monte-Vina	25.49/36.46	14 / 9	8 / 6	6 / 3	\$175,200 / \$126,600	\$6,873	\$3,472	-49.48
Echo Valley / Pesante	25.49/24.31	14 / 17	8 / 12	6 / 5	\$175,200 / \$250,000	\$6,873	\$10,284	49.62
Echo Valley / Summit - NB	25.49/32.07	12 / 14	6 / 5	6 / 9	\$136,200 / \$126,300	\$5,343	\$3,938	-26.30
Echo Valley / Bay Front	25.49/17.06	14 / 3	8 / 1	6 / 2	\$175,200 / \$25,900	\$6,873	\$1,518	-77.91
Echo Valley / Summit - SB	25.49/32.07	14 / 9	8 / 4	6 / 5	\$175,200 / \$94,000	\$6,873	\$2,931	-57.36
Left Turn								
Echo Valley / Black	50.97/72.45	10 / 10	6 / 4	4 / 6	\$129,800 / \$97,200	\$2,547	\$1,342	-47.30
Echo Valley / Dunbarton	50.97/50.52	10 / 10	6 / 4	4 / 6	\$129,800 / \$97,200	\$2,547	\$1,924	-24.46
Echo Valley / Messick #1	50.97/50.97	9 / 8	5 / 2	4 / 6	\$110,300 / \$58,200	\$2,164	\$1,142	-47.23
Echo Valley / Glenwood	50.97/58.25	9 / 17	5 / 7	4 / 10	\$110,300 / \$168,500	\$2,164	\$2,893	33.69
Echo Valley / Sugar Loaf	50.97/58.25	9 / 7	5 / 3	4 / 4	\$110,300 / \$71,300	\$2,164	\$1,224	-43.45
Echo Valley / Glenwood C.	50.97/58.25	9 / 4	5 / 2	4 / 2	\$110,300 / \$45,400	\$2,164	\$779	-64.00

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.30: Comparison of Accident Cost for Four-Lane Narrow Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Crazy Horse / Blackie	25.49/24.31	14 / 13	7 / 6	7 / 7	\$158,900 / \$139,400	\$6,234	\$5,734	-8.02
Crazy Horse / Monte-Vina	25.49/36.46	14 / 9	7 / 6	7 / 3	\$158,900 / \$126,600	\$6,234	\$3,472	-44.30
Crazy Horse / Pesante	25.49/24.31	14 / 17	7 / 12	7 / 5	\$158,900 / \$250,000	\$6,234	\$10,284	64.97
Crazy Horse / Summit - NB	25.49/32.07	12 / 14	6 / 5	6 / 9	\$136,200 / \$126,300	\$5,343	\$3,938	-26.30
Crazy Horse / Bay Front	25.49/17.06	12 / 3	6 / 1	6 / 2	\$136,200 / \$25,900	\$5,343	\$1,518	-71.59
Crazy Horse / Summit - SB	25.49/32.07	12 / 9	6 / 4	6 / 5	\$136,200 / \$94,000	\$5,343	\$2,931	-45.14
Left Turn								
Crazy Horse / Black	50.97/72.45	26 / 10	10 / 4	16 / 6	\$246,200 / \$97,200	\$4,830	\$1,342	-72.22
Crazy Horse / Dunbarton	50.97/50.52	26 / 10	10 / 4	16 / 6	\$246,200 / \$97,200	\$4,830	\$1,924	-60.17
Crazy Horse / Messick #1	50.97/50.97	26 / 8	10 / 2	16 / 6	\$246,200 / \$58,200	\$4,830	\$1,142	-76.36
Crazy Horse / Glenwood	50.97/58.25	26 / 17	10 / 7	16 / 10	\$246,200 / \$168,500	\$4,830	\$2,893	-40.11
Crazy Horse / Sugar Loaf	50.97/58.25	25 / 7	10 / 3	15 / 4	\$243,000 / \$71,300	\$4,768	\$1,224	-74.33
Crazy Horse / Glenwood C.	50.97/58.25	25 / 4	10 / 2	15 / 2	\$243,000 / \$45,400	\$4,768	\$779	-83.66

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.31: Comparison of Accident Cost for Four-Lane Narrow Median Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Bloomfield / SR 183 & I	5.82/15.57	9 / 19	6 / 9	3 / 10	\$126,600 / \$207,500	\$21,753	\$13,327	-38.73
Bloomfield / Uvas	5.82/3.08	9 / 0	6 / 0	3 / 0	\$126,600 / \$0	\$21,753	\$0	-100
Bloomfield / Los Cameros	5.82/10.13	8 / 0	5 / 0	3 / 0	\$107,100 / \$0	\$18,402	\$0	-100
Bloomfield / Cuttings Wharf	5.82/10.95	7 / 1	4 / 1	3 / 0	\$87,600 / \$19,500	\$15,052	\$1,781	-88.17
Bloomfield / Nicolaus	5.82/4.29	7 / 0	4 / 0	3 / 0	\$87,600 / \$0	\$15,052	\$0	-100
Bloomfield / Madison	5.82/8.60	7 / 9	4 / 5	3 / 4	\$87,600 / \$110,300	\$15,052	\$12,826	-14.79
Left Turn								
Bloomfield / Salinas	11.64/31.17	13 / 35	6 / 12	7 / 23	\$139,400 / \$307,600	\$11,976	\$9,868	-17.60
Bloomfield / Torero	11.64/21.43	12 / 6	6 / 3	6 / 3	\$136,200 / \$6,8100	\$11,701	\$3,178	-72.84
Bloomfield / Cathedral Oak	11.64/24.22	12 / 2	6 / 0	6 / 2	\$136,200 / \$6,400	\$11,701	\$264	-97.74
Bloomfield / Bit	11.64/20.86	11 / 3	5 / 3	6 / 0	\$116,700 / \$38,500	\$10,026	\$2,689	-73.18
Bloomfield / Molera	11.64/31.90	11 / 4	5 / 3	6 / 1	\$116,700 / \$61,700	\$10,026	\$1,856	-81.49
Bloomfield / Moss Landing	11.64/33.54	11 / 1	5 / 0	6 / 1	\$116,700 / \$3,200	\$10,026	\$95	-99.05
Bloomfield / Jensen	11.64/33.36	11 / 8	5 / 3	6 / 5	\$116,700 / \$74,500	\$10,026	\$2,233	-77.73
Bloomfield / Meridian	11.64/24.22	11 / 6	5 / 2	6 / 4	\$116,700 / \$51,800	\$10,026	\$2,139	-78.67

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.32: Comparison of Accident Cost for Two-Lane Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Watsonville / SR 183 & I	2.68/15.57	6 / 19	4 / 9	2 / 10	\$84,400 / \$207,500	\$31,493	\$13,327	-57.68
Watsonville / Uvas	2.68/3.08	6 / 0	4 / 0	2 / 0	\$84,400 / \$0	\$31,493	\$0	-100
Watsonville / Los Cameros	2.68/10.13	6 / 0	4 / 0	2 / 0	\$84,400 / \$0	\$31,493	\$0	-100
Watsonville / Cuttings Wharf	2.68/10.95	6 / 1	4 / 1	2 / 0	\$84,400 / \$19,500	\$31,493	\$1,781	-94.34
Watsonville / Nicolaus	2.68/4.29	6 / 0	4 / 0	2 / 0	\$84,400 / \$0	\$31,493	\$0	-100
Watsonville / Madison	2.68/8.60	6 / 9	4 / 5	2 / 4	\$84,400 / \$110,300	\$31,493	\$12,826	-59.27
Left Turn								
Watsonville / Salinas	5.37/31.17	7 / 35	5 / 12	2 / 23	\$103,900 / \$307,600	\$19,348	\$9,868	-49.00
Watsonville / Torero	5.37/21.43	7 / 6	5 / 3	2 / 3	\$103,900 / \$68,100	\$19,348	\$3,178	-83.57
Watsonville / Cathedral Oak	5.37/24.22	7 / 2	5 / 0	2 / 2	\$103,900 / \$6,400	\$19,348	\$264	-98.63
Watsonville / Bit	5.37/20.86	7 / 3	5 / 3	2 / 0	\$103,900 / \$58,500	\$19,348	\$2,689	-86.10
Watsonville / Molera	5.37/31.90	7 / 4	5 / 3	2 / 1	\$103,900 / \$61,700	\$19,348	\$1,856	-90.41
Watsonville / Moss Landing	5.37/33.54	7 / 1	5 / 0	2 / 1	\$103,900 / \$3,200	\$19,348	\$95	-99.51
Watsonville / Jensen	5.37/33.36	7 / 8	5 / 3	2 / 5	\$103,900 / \$74,500	\$19,348	\$2,233	-88.46
Watsonville / Meridian	5.37/24.22	7 / 6	5 / 2	2 / 4	\$103,900 / \$51,800	\$19,348	\$2,139	-88.94

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.33: Comparison of Accident Cost for Two-Lane Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Espinosa / SR 183 & 1	8.38/15.57	7/19	1/9	6/10	\$38,700 / \$207,500	\$4,618	\$13,327	188.58
Espinosa / Uvas	8.38/3.08	7/0	1/0	6/0	\$38,700 / \$0	\$4,618	\$0	-100
Espinosa / Los Cameros	8.38/10.13	7/0	1/0	6/0	\$38,700 / \$0	\$4,618	\$0	-100
Espinosa / Cuttings Wharf	8.38/10.95	7/1	1/1	6/0	\$38,700 / \$19,500	\$4,618	\$1,781	-61.43
Espinosa / Nicolaus	8.38/4.29	7/0	1/0	6/0	\$38,700 / \$0	\$4,618	\$0	-100
Espinosa / Madison	8.38/8.60	7/9	1/5	6/4	\$38,700 / \$110,300	\$4,618	\$12,826	177.72
Left Turn								
Espinosa / Salinas	16.75/31.17	8/35	1/12	7/23	\$41,900 / \$307,600	\$2,501	\$9,868	294.56
Espinosa / Tonero	16.75/21.43	8/6	1/3	7/3	\$41,900 / \$68,100	\$2,501	\$3,178	27.07
Espinosa / Cathedral Oak	16.75/24.22	8/2	1/0	7/2	\$41,900 / \$6,400	\$2,501	\$264	-89.43
Espinosa / Bit	16.75/20.86	8/3	1/3	7/0	\$41,900 / \$58,500	\$2,501	\$2,689	7.52
Espinosa / Molera	16.75/31.90	8/4	1/3	7/1	\$41,900 / \$61,700	\$2,501	\$1,856	-25.79
Espinosa / Moss Landing	16.75/33.54	8/1	1/0	7/1	\$41,900 / \$3,200	\$2,501	\$95	-96.20
Espinosa / Jensen	16.75/33.36	8/8	1/3	7/5	\$41,900 / \$74,500	\$2,501	\$2,233	-10.72
Espinosa / Meridian	16.75/24.22	8/6	1/2	7/4	\$41,900 / \$51,800	\$2,501	\$2,139	-14.47

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.34: Comparison of Accident Cost for Two-Lane Highways - All Accidents

Location (w/o vs. w)	Volume (MV) (w/o vs. w) (a)	Total Accidents (w/o vs. w)	Injury Accidents (w/o vs. w)	PDO Accidents (w/o vs. w)	Total Accident Cost (w/o vs. w)	Cost per MV w/o Accel. Lane	Cost per MV w/ Accel. Lane	Difference (%)
Right Turn								
Fairview / SR 183 & 1	4.49/15.57	10 / 19	4 / 9	6 / 10	\$97,200 / \$207,500	\$21,648	\$13,327	-38.44
Fairview / Uvas	4.49/3.08	10 / 0	4 / 0	6 / 0	\$97,200 / \$0	\$21,648	\$0	-100
Fairview / Los Cameros	4.49/10.13	10 / 0	4 / 0	6 / 0	\$97,200 / \$0	\$21,648	\$0	-100
Fairview / Cuttings Wharf	4.49/10.95	10 / 1	4 / 1	6 / 0	\$97,200 / \$19,500	\$21,648	\$1,781	-91.77
Fairview / Nicolaus	4.49/4.29	10 / 0	4 / 0	6 / 0	\$97,200 / \$0	\$21,648	\$0	-100
Fairview / Madison	4.49/8.60	10 / 9	4 / 5	6 / 4	\$97,200 / \$110,300	\$21,648	\$12,826	-40.75
Left Turn								
Fairview / Salinas	8.98/31.17	21 / 35	11 / 12	10 / 23	\$246,500 / \$307,600	\$27,450	\$9,869	-64.05
Fairview / Torero	8.98/21.43	21 / 6	11 / 3	10 / 3	\$246,500 / \$68,100	\$27,450	\$3,178	-88.42
Fairview / Cathedral Oak	8.98/24.22	21 / 2	11 / 0	10 / 2	\$246,500 / \$6,400	\$27,450	\$264	-99.04
Fairview / Bit	8.98/20.86	21 / 3	11 / 3	10 / 0	\$246,500 / \$58,300	\$27,450	\$2,689	-90.20
Fairview / Molera	8.98/31.90	21 / 4	11 / 3	10 / 1	\$246,500 / \$61,700	\$27,450	\$1,856	-93.24
Fairview / Moss Landing	8.98/33.54	21 / 1	11 / 0	10 / 1	\$246,500 / \$3,200	\$27,450	\$95	-99.65
Fairview / Jensen	8.98/33.36	21 / 8	11 / 3	10 / 5	\$246,500 / \$74,300	\$27,450	\$2,233	-91.87
Fairview / Meridian	8.98/24.22	21 / 6	11 / 2	10 / 4	\$246,500 / \$51,800	\$27,450	\$2,139	-92.21

(a) For right turn acceleration lanes the volume is for one direction only.

TABLE 5.35: Summary of Average Accident Costs According to Sites Without Acceleration Lane - All Accidents

Control Sites vs. Sites w/ Accel. Lane (w/o vs. w)	Total Acc. Cost of All Sites w/o Accel. Lane	Total Acc. Cost of All Sites w/ Accel. Lane	Total Acc. Cost per MV of All Sites w/o Accel. Lane	Total Acc. Cost per MV of All Sites w/ Accel. Lane	Difference (%)
Right Turn:					
4-Lane-Wide Median					
Spence / All Sites w/ Accel. Lane	\$318,100	\$295,400	\$5,734	\$4,365	-23.87
McCloskey / All Sites w/ Accel. Lane	\$872,800	\$295,400	\$29,607	\$4,365	-85.26
Yerba Buena / All Sites w/ Accel. Lane	\$154,800	\$295,400	\$4,500	\$4,365	-3.00
Average	\$1,345,700	\$886,200	\$11,274	\$3,103	-72.48
4-Lane-Narrow Median					
Tustin / All Sites w/ Accel. Lane	\$610,000	\$496,200	\$5,983	\$4,703	-21.39
Echo Valley / All Sites w/ Accel. Lane	\$636,200	\$496,200	\$6,240	\$4,703	-24.63
Crazy Horse / All Sites w/ Accel. Lane	\$567,500	\$496,200	\$5,566	\$4,703	-15.51
Average	\$1,813,700	\$1,488,600	\$5,929	\$4,703	-20.69
2-Lane					
Bloomfield / All Sites w/ Accel. Lane	\$623,100	\$337,300	\$17,844	\$6,410	-64.08
Watsonville / All Sites w/ Accel. Lane	\$506,400	\$337,300	\$31,493	\$6,410	-79.65
Espinoso / All Sites w/ Accel. Lane	\$232,200	\$337,300	\$4,618	\$6,410	38.80
Fairview / All Sites w/ Accel. Lane	\$583,200	\$337,300	\$21,648	\$6,410	-70.39
Average	\$1,944,900	\$1,349,200	\$15,168	\$6,410	-57.74
Average for Right Turns	\$5,104,300	\$3,658,800	\$9,223	\$4,502	-51.18
Left Turn:					
4-Lane-Wide Median					
Spence / All Sites w/ Accel. Lane	\$298,000	\$373,100	\$3,011	\$3,794	25.99
McCloskey / All Sites w/ Accel. Lane	\$869,600	\$373,100	\$14,739	\$3,794	-74.26
Yerba Buena / All Sites w/ Accel. Lane	\$252,000	\$373,100	\$3,665	\$3,794	3.52
Average	\$1,419,600	\$1,119,300	\$6,261	\$3,794	-39.41
4-Lane-Narrow Median					
Tustin / All Sites w/ Accel. Lane	\$576,800	\$537,800	\$1,886	\$1,542	-18.22
Echo Valley / All Sites w/ Accel. Lane	\$700,800	\$537,800	\$2,292	\$1,542	-32.69
Crazy Horse / All Sites w/ Accel. Lane	\$1,470,800	\$537,800	\$4,809	\$1,542	-67.93
Average	\$2,751,600	\$1,613,400	\$2,999	\$1,542	-48.57
2-Lane					
Bloomfield / All Sites w/ Accel. Lane	\$1,158,300	\$631,800	\$12,439	\$2,862	-76.99
Watsonville / All Sites w/ Accel. Lane	\$831,200	\$631,800	\$19,348	\$2,862	-85.21
Espinoso / All Sites w/ Accel. Lane	\$335,200	\$631,800	\$2,501	\$2,862	14.42
Fairview / All Sites w/ Accel. Lane	\$1,972,000	\$631,800	\$27,450	\$2,862	-89.57
Average	\$4,296,700	\$2,527,200	\$12,566	\$2,862	-77.22
Average for Left Turns	\$8,112,900	\$5,029,500	\$5,459	\$2,261	-58.58

TABLE 5.36: Summary of Average Accident Costs According to Sites With Right Turn Acceleration Lane - All Accidents

Control Sites vs. Sites w/ Accel. Lane (w/o vs. w)	Total Acc. Cost of All Sites w/o Accel. Lane	Total Acc. Cost of All Sites w/ Accel. Lane	Total Acc. Cost per MV of All Sites w/o Accel. Lane	Total Acc. Cost per MV of All Sites w/ Accel. Lane	Difference (%)
4-Lane-Wide Median					
All Control Sites / Espinosa	\$363,200	\$64,900	\$12,172	\$2,680	-77.98
All Control Sites / Castro Valley	\$340,500	\$84,400	\$11,411	\$3,504	-69.29
All Control Sites / Ocean	\$321,000	\$120,200	\$10,757	\$4,028	-62.56
All Control Sites / Bay Front	\$321,000	\$25,900	\$10,757	\$1,518	-85.89
Average	\$1,345,700	\$295,400	\$10,470	\$3,103	-70.37
4-Lane-Narrow Median					
All Control Sites / Pesante	\$486,600	\$250,000	\$6,363	\$10,284	61.61
All Control Sites / Summit - NB	\$424,900	\$126,300	\$5,556	\$3,938	-29.13
All Control Sites / Bay Front	\$463,900	\$25,900	\$6,066	\$1,518	-74.98
All Control Sites / Summit - SB	\$463,900	\$94,000	\$6,066	\$2,931	-51.68
Average	\$1,375,400	\$402,200	\$6,013	\$4,703	-21.79
2-Lane					
All Control Sites / SR 183	\$346,900	\$207,500	\$16,233	\$13,327	-17.90
All Control Sites / Uvas	\$346,900	\$0	\$16,233	\$0	-100.00
All Control Sites / Los Canteros	\$327,400	\$0	\$15,321	\$0	-100.00
All Control Sites / Cuttings Wharf	\$307,900	\$19,500	\$14,408	\$1,781	-87.64
All Control Sites / Nicolaus	\$307,900	\$0	\$14,408	\$0	-100.00
All Control Sites / Madison	\$307,900	\$110,300	\$14,408	\$12,826	-10.98
Average	\$1,944,900	\$337,300	\$15,168	\$6,410	-57.74
Average for Right Turns	\$5,033,900	\$1,128,900	\$9,095	\$4,456	-51.01

TABLE 5.37: Summary of Average Accident Costs According to Sites With Left Turn Acceleration Lane - All Accidents

Control Sites vs. Sites w/ Accel. Lane (w/o vs. w)	Total Acc. Cost of All Sites w/o Accel. Lane	Total Acc. Cost of All Sites w/ Accel. Lane	Total Acc. Cost per MV of All Sites w/o Accel. Lane	Total Acc. Cost per MV of All Sites w/ Accel. Lane	Difference (%)
4-Lane-Wide Median					
All Control Sites / Bell Creek	\$356,500	\$3,200	\$5,974	\$170	-97.15
All Control Sites / Tower	\$356,500	\$74,500	\$5,974	\$2,005	-66.44
All Control Sites / Solano	\$356,500	\$61,700	\$5,974	\$2,955	-50.53
All Control Sites / Oak Knoll	\$356,500	\$233,700	\$5,974	\$10,880	82.14
Average	\$1,426,000	\$373,100	\$5,947	\$3,794	-36.20
4-Lane-Narrow Median					
All Control Sites / Black	\$473,200	\$97,200	\$3,095	\$1,342	-56.63
All Control Sites / Dunbarton	\$473,200	\$97,200	\$3,095	\$1,924	-37.83
All Control Sites / Messick #1	\$453,700	\$58,200	\$2,967	\$1,142	-61.51
All Control Sites / Glenwood	\$450,500	\$168,500	\$2,946	\$2,893	-1.80
All Control Sites / Sugar Loaf	\$447,300	\$71,300	\$2,925	\$1,224	-58.16
All Control Sites / Glenwood Cutoff	\$447,300	\$45,400	\$2,925	\$779	-73.37
Average	\$2,745,200	\$537,800	\$2,992	\$1,542	-48.45
2-Lane					
All Control Sites / Salinas	\$512,200	\$307,600	\$11,984	\$9,868	-17.66
All Control Sites / Torero	\$509,000	\$68,100	\$11,909	\$3,178	-73.31
All Control Sites / Cathedral Oak	\$509,000	\$6,400	\$11,909	\$264	-97.78
All Control Sites / Bit	\$489,500	\$58,500	\$11,453	\$2,689	-76.52
All Control Sites / Molera	\$489,500	\$61,700	\$11,453	\$1,856	-83.79
All Control Sites / Moss Landing	\$489,500	\$3,200	\$11,453	\$95	-99.17
All Control Sites / Jensen	\$489,500	\$74,500	\$11,453	\$2,233	-80.50
All Control Sites / Meridian	\$489,500	\$51,800	\$11,453	\$2,139	-81.32
Average	\$3,977,700	\$631,800	\$11,633	\$2,862	-75.40
Average for Left Turns	\$8,266,500	\$1,542,700	\$5,518	\$2,310	-58.13

The sites at Elverta, Catlett, Blackie and Montevina were not included in the averages, since both right and left turn acceleration lanes were present in the same direction and it was not possible to isolate the effect of the acceleration lane studied. Notwithstanding this problem, accident reductions at these sites are still indicative of benefits resulting from the construction of acceleration lanes.

The accident cost for each site with an acceleration lane was calculated using the Caltrans values for accident costs applicable to rural areas.

These accident costs are as follows:

Fatal:	\$782,000
Injury:	\$19,500
PDO:	\$3,200

In the following sections, comparisons will be made in terms of intersection category, left turn versus right turns and type of highway. Also, the performances of different lengths of acceleration lanes were evaluated in terms of accident rates and costs, followed by an economic analysis.

Analysis Within Intersection Categories

With few exceptions, the sites with acceleration lanes had lower accident rates and accident costs per million vehicles. Only the Pesante (for right turns) and Oak Knoll (for left turns) intersections

did not yield accident rate and cost reductions. Physical inspection of the Pesante site indicated that turning vehicles may experience limited sight distance.

The reduction in rates and costs appear to be comparable for all categories, with the exception of the four-lane narrow median category for right turning vehicles. This is probably an aberration, since there should not be a substantial difference between the operation of acceleration lanes for right turns in the four-lane wide versus four-lane narrow median categories.

From the results, the conclusion can be drawn that the acceleration lanes do provide a safety benefit for all categories.

Analysis of Right Turn Versus Left Turn Movements

The summaries of average accident rates and costs indicate that left turn acceleration lanes yielded slightly higher decreases than the right turn acceleration lanes. This appears logical since the left turn maneuver is more dangerous than the right turn movement and the provision of the acceleration lane, even if just used as a refuge, should have an appreciable effect.

Comparison of Performance of Acceleration Lanes by Type of Highway

Based on the average accident rates and costs, it appears that the greatest benefit for right turn acceleration lanes can be obtained in

the four-lane wide median category. It should, however, not be much different from the narrow median category, since the width of median should not make a big difference. It is noteworthy that the differences between the design (or average highway) speeds of the sites with acceleration lanes and the control sites are less for the narrow median category than for the wide median category.

Greater benefits appear to be had for the two-lane category than for the four-lane narrow median category, which appears logical. The lower benefits as compared to the wide median category is again unexpected. Based on the absolute value of accident cost reduction per million vehicles, however, the two-lane category yielded higher benefits.

In the case of left turn acceleration lanes, the results appear to follow logic. The two-lane category showed the greatest benefit in percentage terms and in absolute cost terms, followed by the four-lane narrow median category.

Accident Rates and Costs Versus Acceleration Lane Length

The accident rates versus acceleration lane length are shown in Figures 5.3 through 5.8 and the cost versus length in Figures 5.9 through 5.14. For the control sites, the average accident rates and costs are as shown in Tables 5.23, 5.24, 5.35 and 5.36.

FIGURE 5.3: Accident Rates vs Acceleration Lane Length for Four-Lane Wide Median Highways - Right Turn

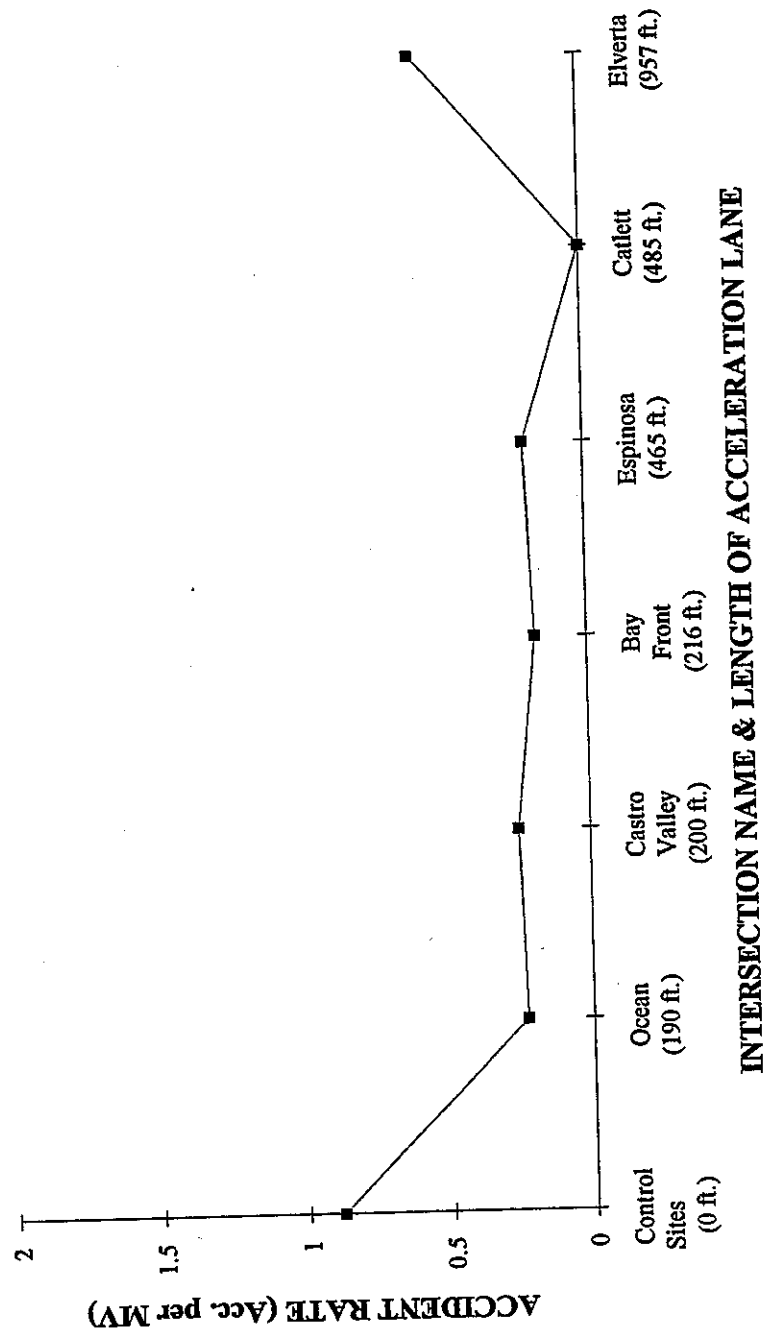


FIGURE 5.4: Accident Rates vs Acceleration Lane Length for Four-Lane Narrow Median Highways - Right Turn

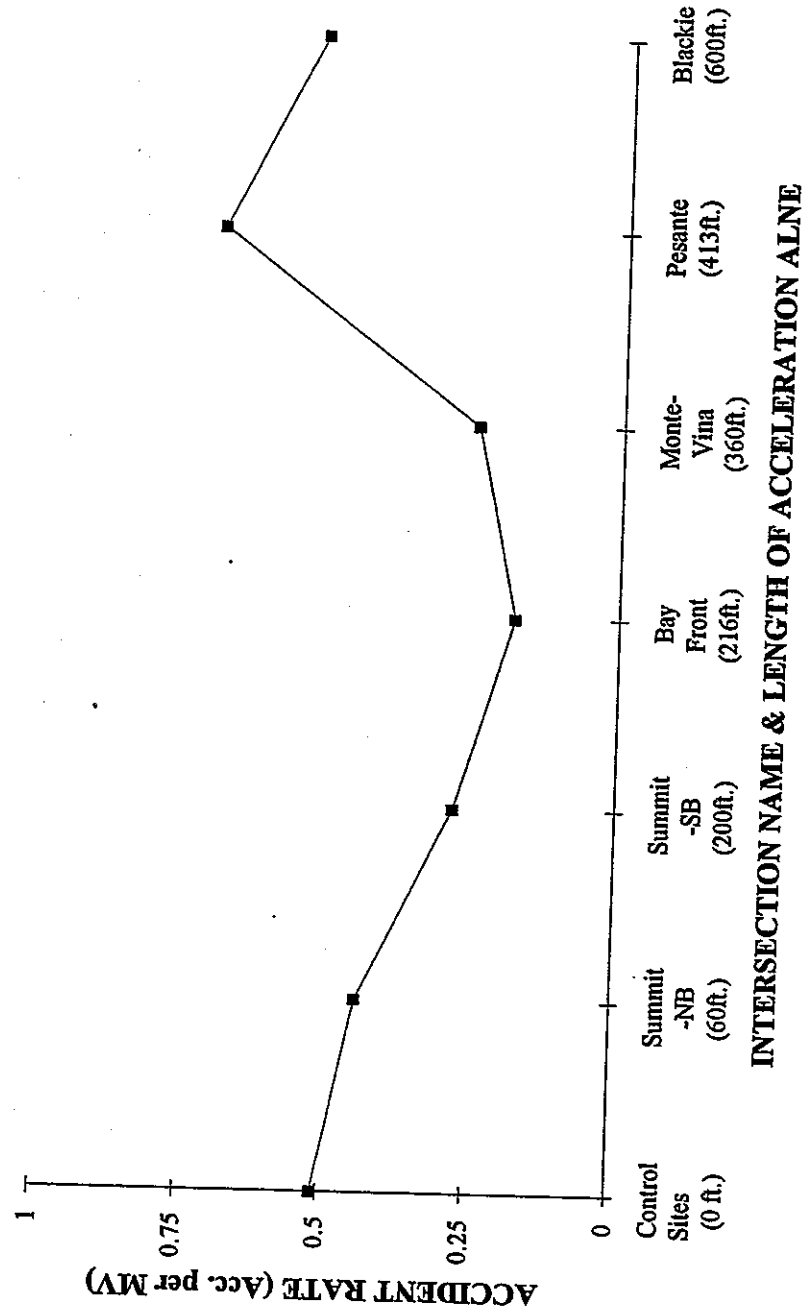
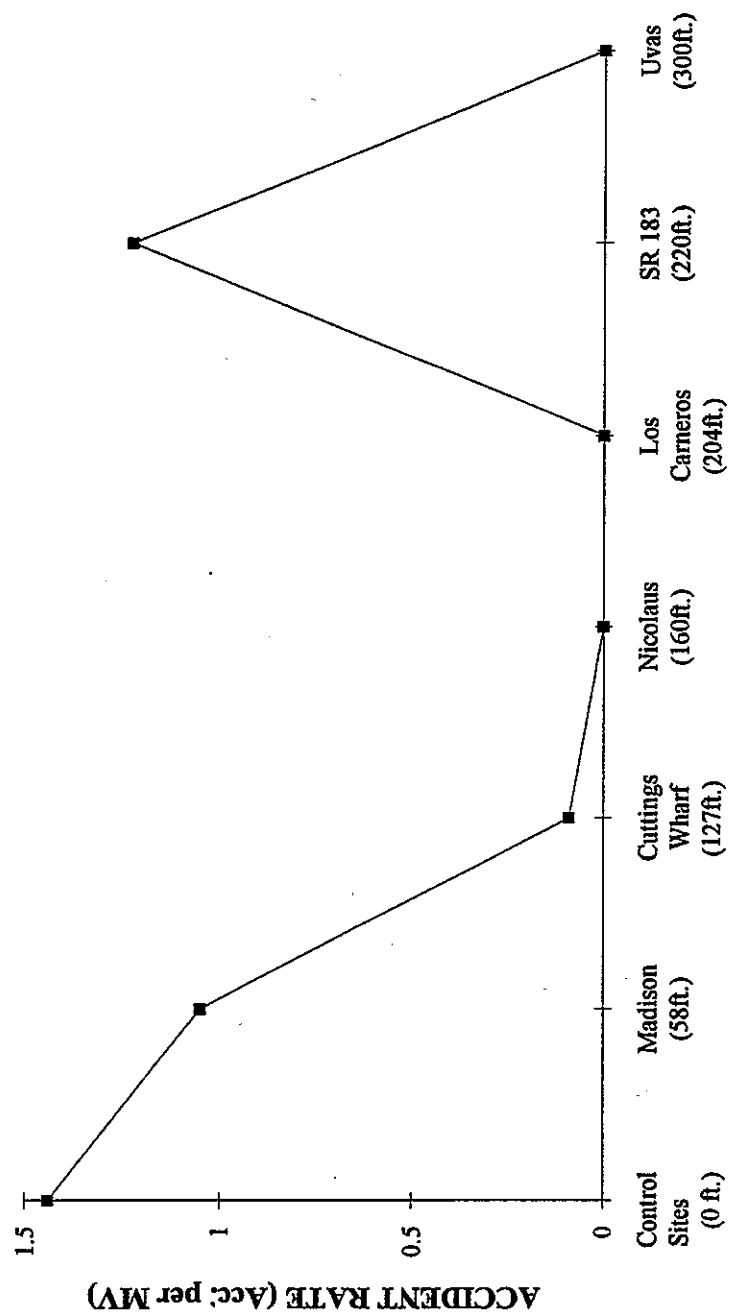
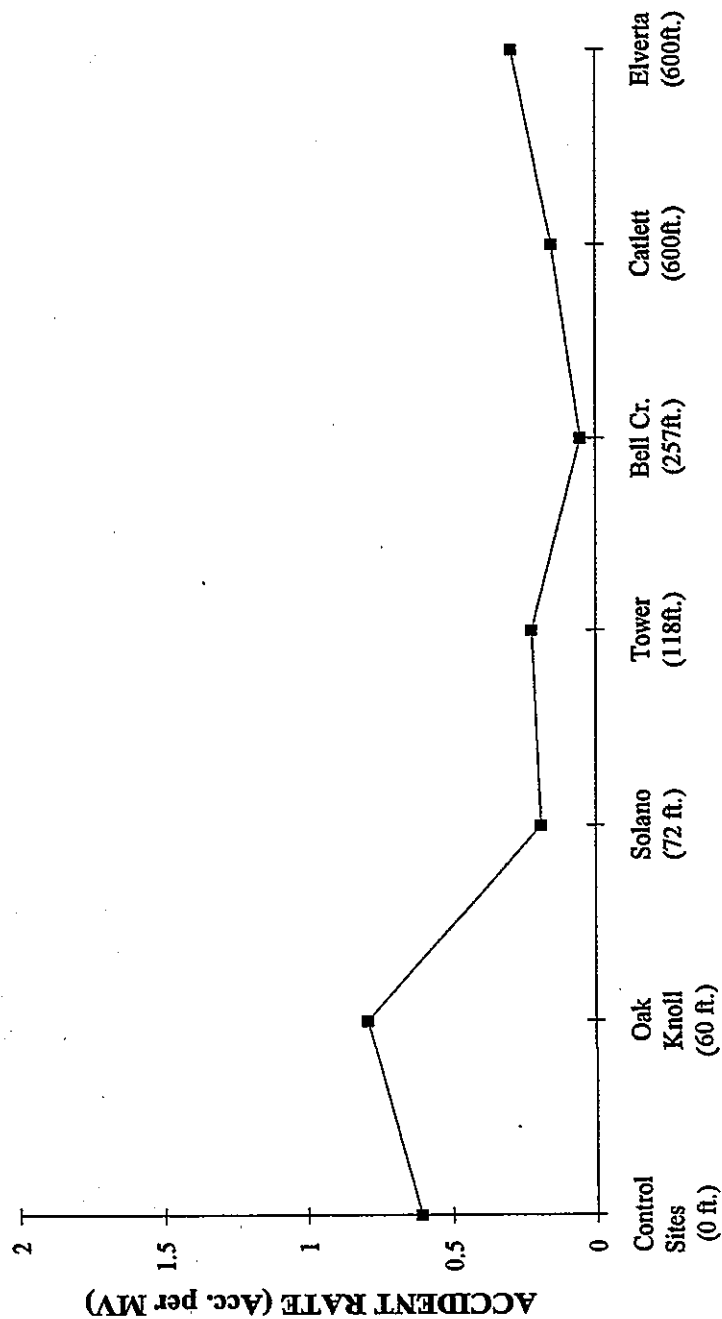


FIGURE 5.5: Accident Rates vs Acceleration Lane Length for Two-Lane Highways - Right Turn



INTERSECTION NAME & LENGTH OF ACCELERATION LANE

FIGURE 5.6: Accident Rates vs Acceleration Lane Length for Four-Lane Wide Median Highways - Left Turn



INTERSECTION NAME & LENGTH OF ACCELERATION LANE

FIGURE 5.7: Accident Rates vs Acceleration Lane Length for Four-Lane Narrow Median Highways - Left Turn

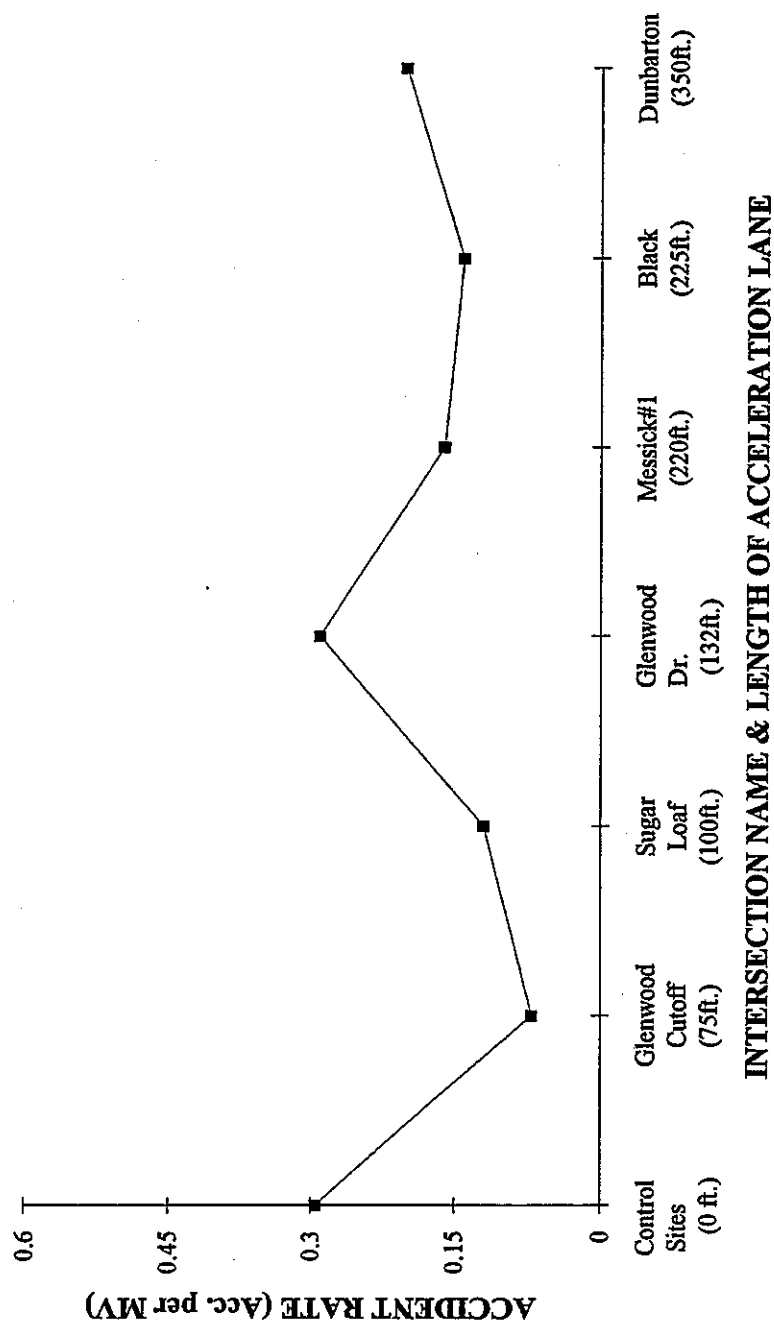


FIGURE 5.3: Accident Rates vs Acceleration Lane Length for Two-Lane Highways - Left Turn

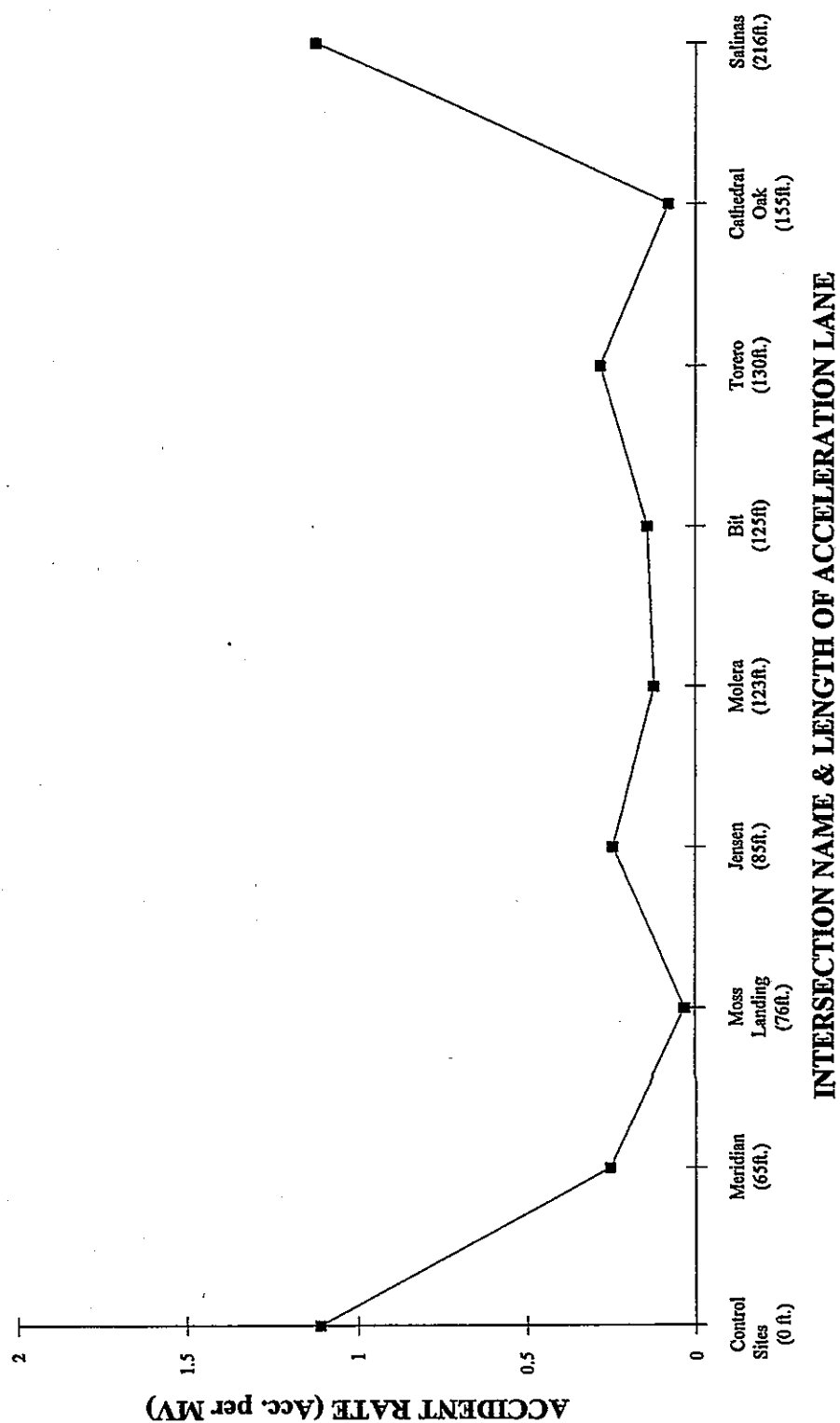


FIGURE 5.9: Accident Cost vs Acceleration Lane Length for Four-Lane Wide Median Highways - Right Turn

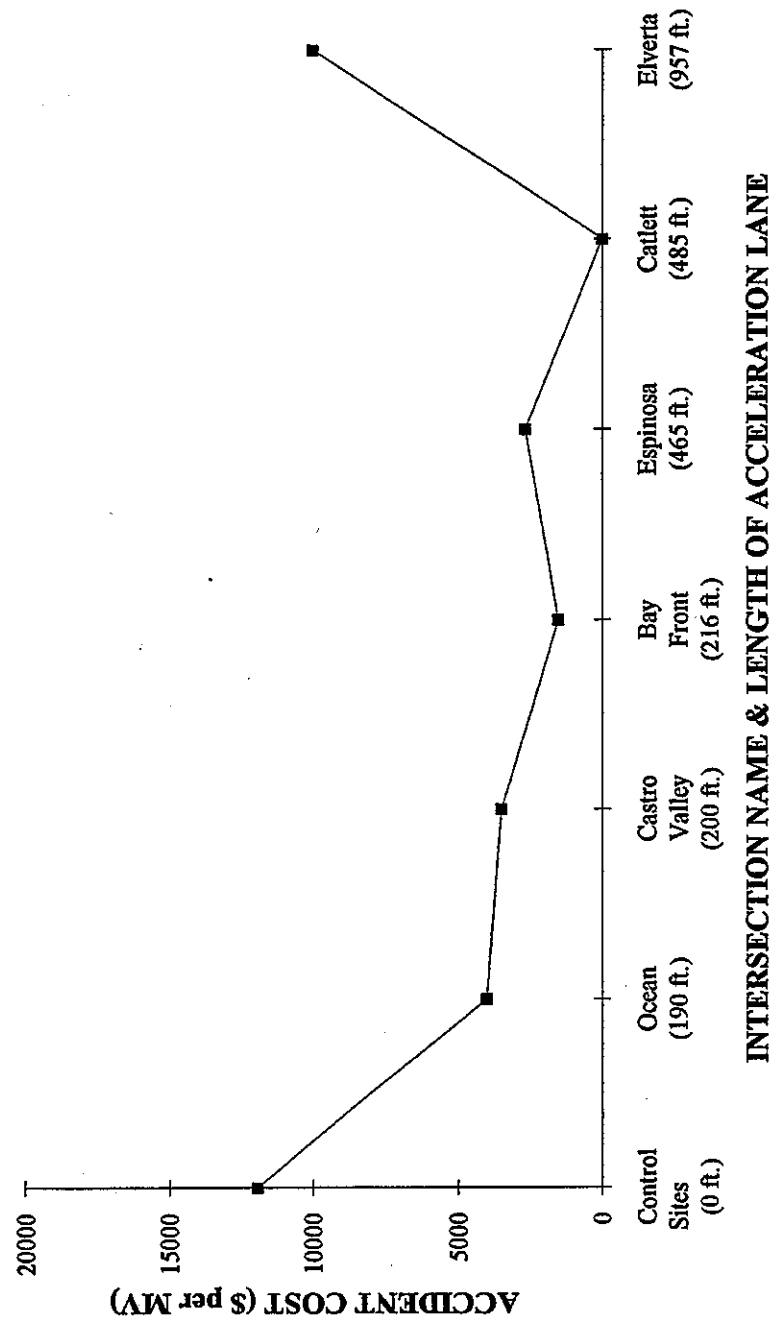


FIGURE 5.10: Accident Cost vs Acceleration Lane Length for Four-Lane Narrow Median Highways - Right Turn

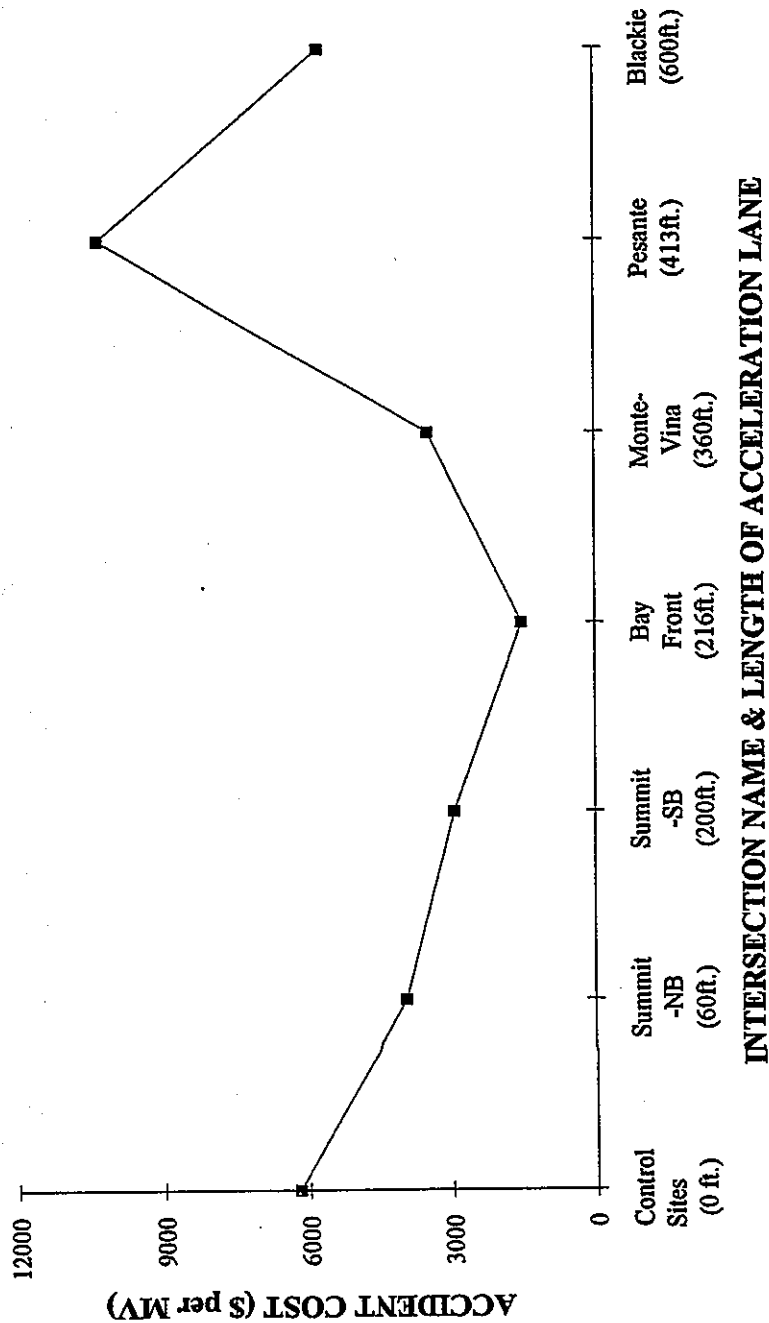


FIGURE 5.11: Accident Cost vs Acceleration Lane Length for Two-Lane Highways - Right Turn

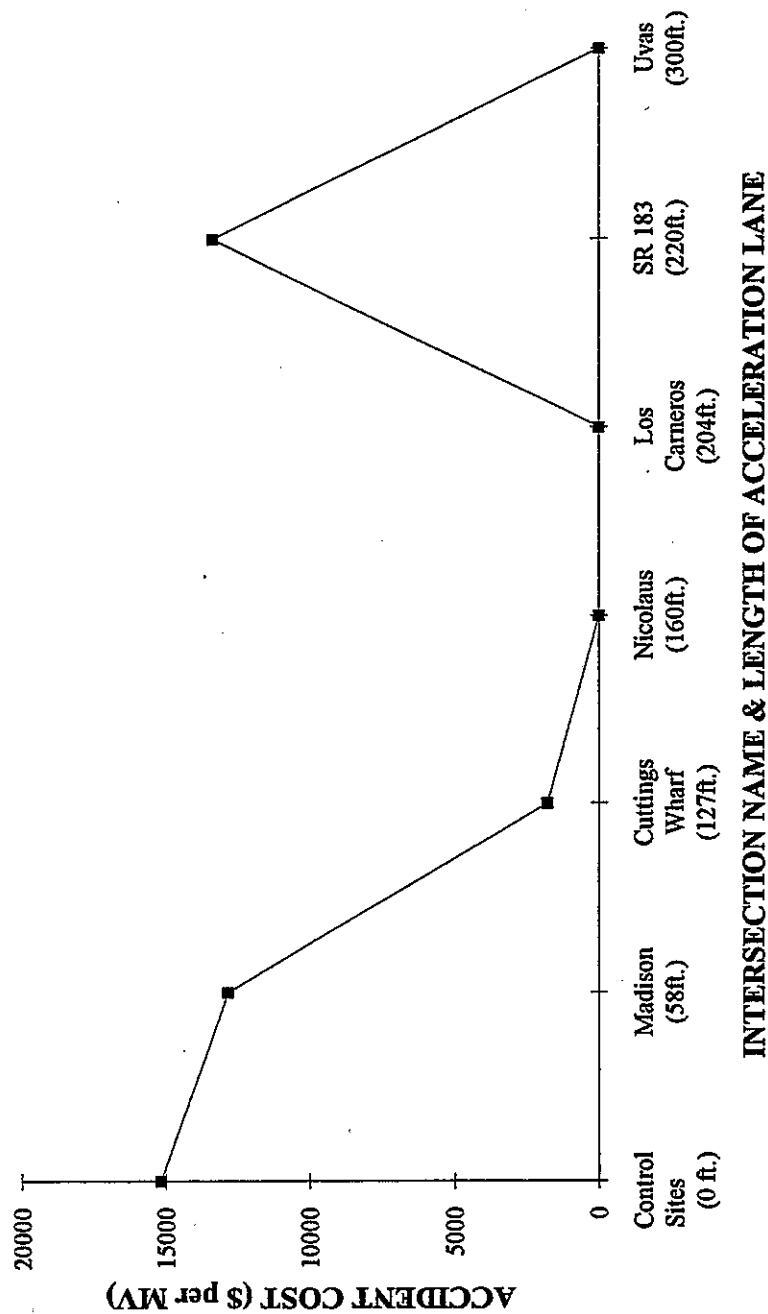


FIGURE 5.12: Accident Cost vs Acceleration Lane Length for Four-Lane Wide Median Highways - Left Turn

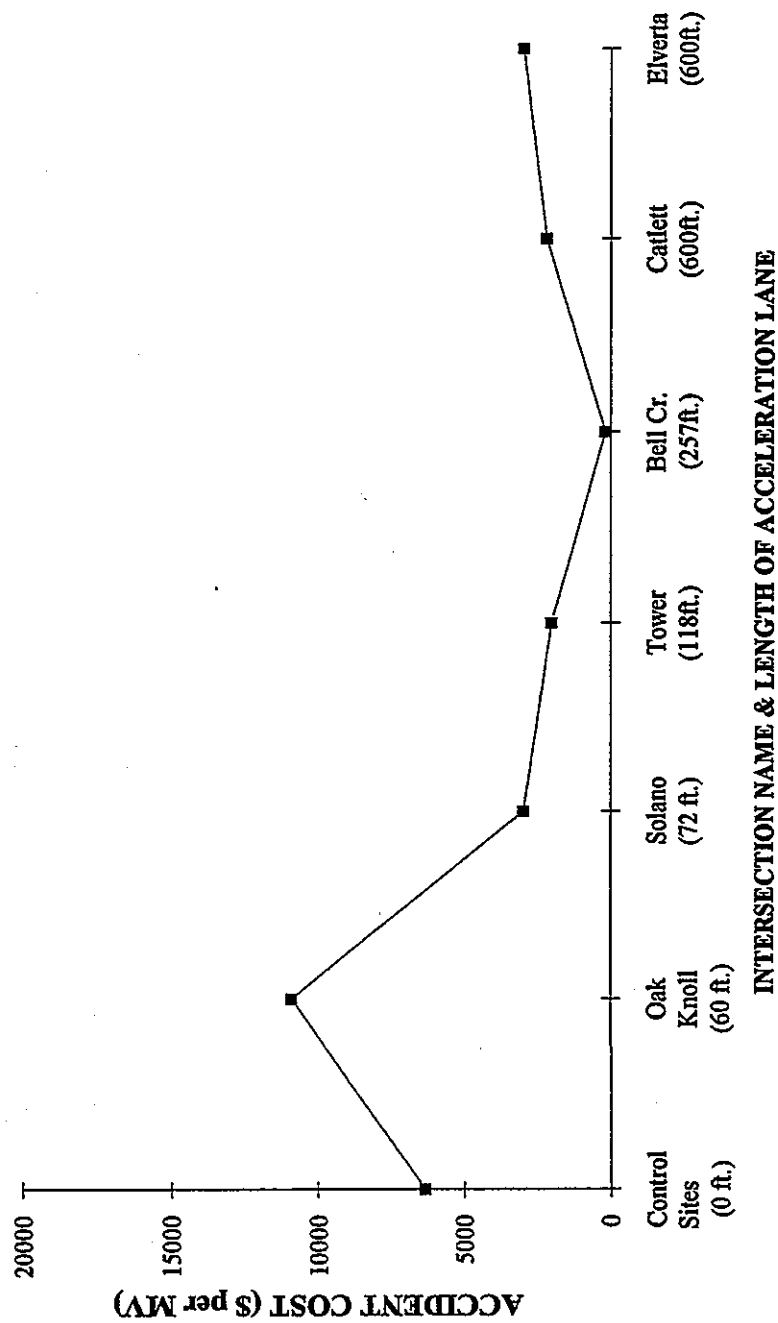


FIGURE 5.13: Accident Cost vs Acceleration Lane Length for Four-Lane Narrow Median Highways - Left Turn

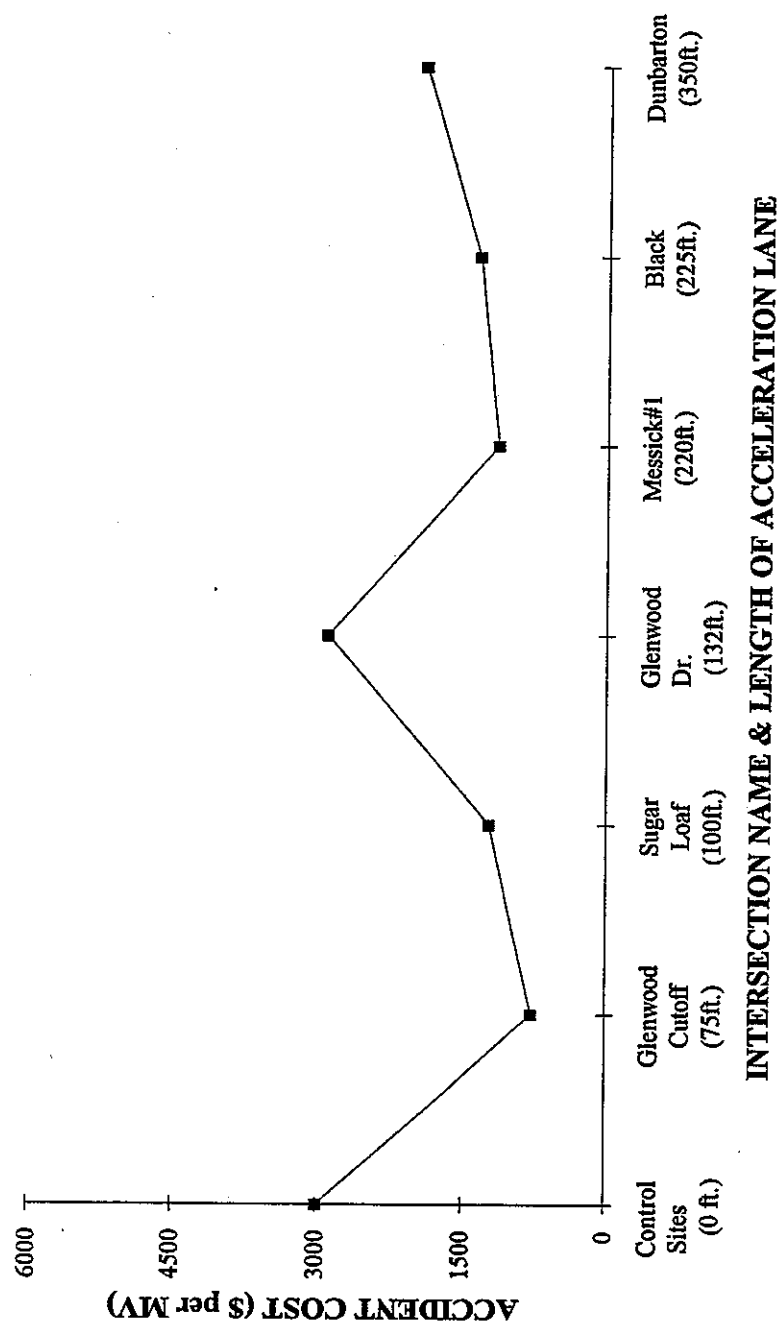
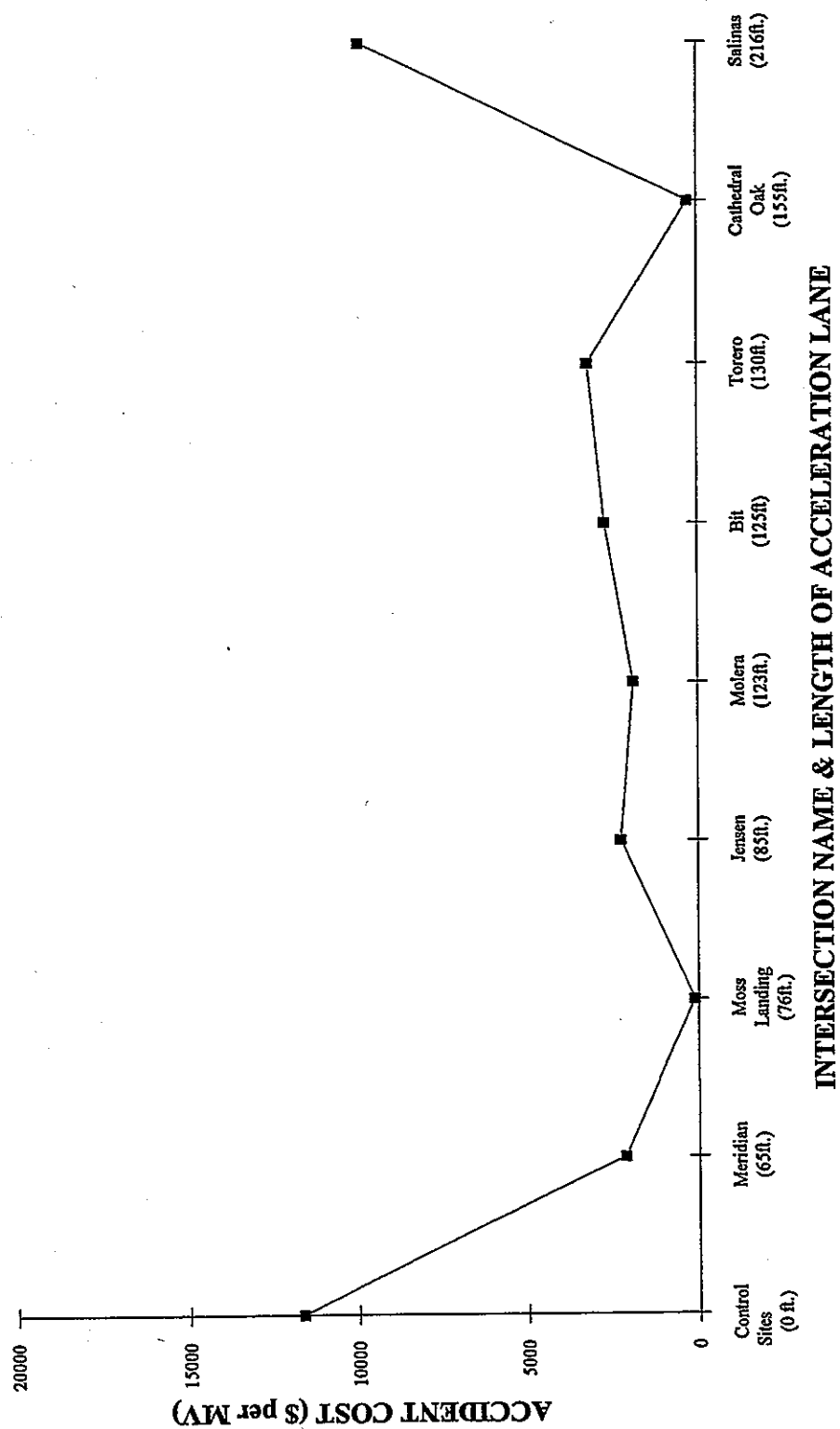


FIGURE 5.14: Accident Cost vs Acceleration Lane Length for Two-Lane Highways - Left Turn



The results show that the accident rates and costs for right turns in the four-lane category decrease up to a length of at least 465 ft., when the sites with more than one acceleration lane (Elverta, Catlett, Montevina and Blackie) are excluded. It is possible that longer lanes will also yield benefits, if the results obtained for the excluded sites were also considered. It is noteworthy that the high accident rate and cost at the Pesante site may be due to the fact that the intersection is at the top of a crest vertical curve. For two-lane highways, benefits were obtained for a length of up to 300 ft.

In the case of left turns for four-lane highways with a wide median, benefits can be obtained by increasing the length to 275 ft., if the results for the Elverta and Catlett sites were excluded. If the latter sites were considered, then it may be concluded that benefits may be obtained for lanes longer than 275 ft.

In the four-lane narrow median category, benefits appear to be possible for lengths up to 350 ft. It is possible that a length beyond 350 ft., which is the maximum studied for this project, may be beneficial. It should be noted that the sharp increase in accident rate and cost for the Glenwood site may be due to limited sight distance.

For two-lane highways, benefits were obtained for a length of up to 216 ft. Again, additional benefits may be had for acceleration lanes with lengths greater than those studied. The increased accident rate and cost at the Salinas site may be due to the site being at the top of a crest vertical curve. Another reason may be that the number of lanes

is reduced from four to two lanes just prior to the intersection, in the southbound direction.

It is noteworthy that, although long acceleration lanes yield benefits over the base condition (no acceleration lanes), the greatest reduction in accident rates and costs occur in the 100 to 200 ft. range.

Economic Analysis

An economic analysis was carried out by calculating a benefit/cost ratio. The benefits consisted of accident cost reduction. The costs were determined in terms of construction and maintenance costs in the same way as calculated in Section 4.3 of this report. The results are presented in Tables 5.38 and 5.39. The sites where more than one acceleration lane were present in the same direction were again eliminated.

From the analysis it can be seen that with the exception of the Pesante, Oak Knoll and Glenwood intersections, all sites showed favorable benefit/cost ratios. This result indicates that, in general, for the range of lengths and traffic volumes prevailing at the sites, acceleration lanes yield net benefits. It should be noted that since right of way costs were not included, the benefit/cost ratio may, in reality, be smaller. Nevertheless, the magnitude of the calculated ratios would indicate that the acceleration lanes should yield a net benefit.

TABLE 5.38: Economic Analysis for Right Turn Acceleration Lanes - Safety

Category & Name of Intersection	Length (ft)	Construction Cost	EUAC Construction	Maintenance Cost	Total EUAC	Cost of Diff/MV	MV of Each Year	Total EUAB	B/C Ratio
4-Lane Wide Median									
High Standard									
Espinosa	465	\$83,700	\$6,716.09	\$125.55	\$6,842	\$9,492	8.07	\$76,600	11.20
Low Standard									
Castro Valley	200	\$36,000	\$2,888.64	\$54.00	\$2,943	\$7,907	8.03	\$63,493	21.58
Ocean	190	\$34,200	\$2,744.21	\$51.30	\$2,796	\$6,729	9.95	\$66,954	23.95
Bay Front	216	\$38,880	\$3,119.73	\$58.32	\$3,178	\$9,239	5.69	\$32,570	16.54
4-Lane Narrow Median									
High Standard									
Pesante	413	\$74,340	\$5,965.04	\$111.51	\$6,077	(\$3,921)	8.10	(\$31,760)	-5.23
Low Standard									
Summit-NB	60	\$10,800	\$866.59	\$16.20	\$883	\$1,618	10.69	\$17,296	19.59
Bay Front	216	\$38,880	\$3,119.73	\$58.32	\$3,178	\$4,548	5.69	\$25,878	8.14
Summit-SB	200	\$36,000	\$2,888.64	\$54.00	\$2,943	\$3,135	10.69	\$33,513	11.39
2-Lane									
High Standard									
SR 183	200	\$36,000	\$2,888.64	\$54.00	\$2,943	\$2,906	5.19	\$15,082	5.13
Uvas	300	\$54,000	\$4,332.96	\$81.00	\$4,414	\$16,233	1.03	\$16,720	3.79
Los Carneros	204	\$36,720	\$2,946.41	\$55.08	\$3,001	\$15,321	3.38	\$51,785	17.25
Low Standard									
Cuttings Wharf	127	\$22,860	\$1,834.29	\$34.29	\$1,869	\$12,627	3.65	\$46,089	24.67
Nicolaus	160	\$28,800	\$2,310.91	\$43.20	\$2,354	\$14,408	1.43	\$20,603	8.75
Madison	58	\$10,440	\$837.71	\$15.66	\$853	\$1,582	2.87	\$4,540	5.32

Note: Negative number shown in parentheses.

TABLE 5.39: Economic Analysis for Left Turn Acceleration Lanes - Safety

Category & Name of Intersection	Length (ft)	Construction Cost	EUAC Construction	Maintenance Cost	Total EUAC	Cost of Diff/MV	MV of Each Year	Total EUAB	B/C Ratio
4-Lane Wide Median High Standard									
Bell Cr.	257	\$46,260	\$3,711.90	\$69.39	\$3,781	\$5,804	6.27	\$36,391	9.62
Low Standard									
Tower	118	\$21,240	\$1,704.30	\$31.86	\$1,736	\$3,969	12.39	\$49,176	28.32
Solano	72	\$12,960	\$1,039.91	\$19.44	\$1,059	\$3,019	6.96	\$21,012	19.84
Oak Knoll	60	\$10,800	\$866.59	\$16.20	\$883	(\$4,906)	7.16	(\$35,127)	-39.79
4-Lane Narrow Median High Standard									
Black	225	\$40,500	\$3,249.72	\$60.75	\$3,310 +	\$1,753	24.15	\$42,335	12.79
Dunbarton	350	\$63,000	\$5,055.12	\$94.50	\$5,150	\$1,171	16.84	\$19,720	3.83
Messick #1	220	\$39,600	\$3,177.50	\$59.40	\$3,237	\$1,825	16.99	\$31,007	9.58
Low Standard									
Glenwood	132	\$23,760	\$1,906.50	\$35.64	\$1,942	\$53	19.42	\$1,029	0.53
Sugar Loaf	100	\$18,000	\$1,444.32	\$27.00	\$1,471	\$1,701	19.42	\$33,033	22.45
Glenwood Cutoff	75	\$13,500	\$1,083.24	\$20.25	\$1,103	\$2,146	19.42	\$41,675	37.77
2-Lane High Standard									
Salinas	216	\$38,880	\$3,119.73	\$58.32	\$3,178	\$2,116	10.39	\$21,985	6.92
Torero Rd.	130	\$23,400	\$1,877.62	\$35.10	\$1,913	\$8,731	7.14	\$62,339	32.59
Cathedral Oak	155	\$27,900	\$2,238.70	\$41.85	\$2,281	\$11,645	8.07	\$93,975	41.21
Bit Rd.	125	\$22,500	\$1,805.40	\$33.75	\$1,839	\$8,764	6.95	\$60,910	33.12
Molera	123	\$22,140	\$1,776.51	\$33.21	\$1,810	\$9,597	10.63	\$102,016	56.37
Low Standard									
Moss Landing	76	\$13,680	\$1,097.68	\$20.52	\$1,118	\$11,358	11.18	\$126,982	113.56
Jensen	85	\$15,300	\$1,227.67	\$22.95	\$1,251	\$9,220	11.12	\$102,526	81.98
Meridian	65	\$11,700	\$938.81	\$17.55	\$956	\$9,314	8.07	\$75,164	78.59

Note: Negative number shown in parentheses.

5.4 Conclusions

The characteristics of the acceleration lanes studied and the major conclusions reached, are summarized below:

	<u>ADT</u> (through traffic)	<u>Length</u> (ft.)
Right Turn Lanes		
Four-Lane Highways	19,900 to 64,300	60 to 957
Two-Lane Highways	5,000 to 30,800	58 to 300
Left Turn Lanes		
Four-Lane Highways	14,500 to 64,300	60 to 600
Two-Lane Highways	19,700 to 30,900	65 to 216

1. Left turn acceleration lanes yielded slightly higher decreases in accident rates and costs than right turn acceleration lanes.
2. Based on absolute cost (as opposed to cost per million vehicles), right turn acceleration lanes performed better on two-lane roads than on four-lane roads.
3. For left turns, acceleration lanes on two-lane highways performed better than on four-lane highways, followed by the four-lane narrow median category.
4. The greatest reduction in accident rates and costs occurred for lengths up to approximately 200 ft. Benefits were also found for lengths of 957 ft. for right turns and up to 600 ft. for left turns. It may also be speculated that benefits could be obtained for lengths greater than the latter.

5. It appears that the acceleration lanes studied are economically justified, given a reasonable assumption for discount rate and service life. The cost of right of way was not included. It should be noted that the combination of through traffic and turning traffic volume should be a factor, but these data were not available for all the sites studied during the safety analysis. It may be concluded, however, that the acceleration lanes would be justified for typical combinations of through and turning traffic volumes encountered at these types of intersections. The lowest combinations of through and turning traffic volumes which yielded a favorable benefit/cost ratio were as follows:

	ADT	Turning Flow Rates (vph)
Right Turn Lanes		
Four-Lane Highways	46,500	84
Two-Lane Highways	20,200	69
Left Turn Lanes		
Four-Lane Highways		
Wide Median	33,600	38
Narrow Median	55,600	17
Two-Lane Highways	30,900	49

6. SUMMARY OF MAJOR CONCLUSIONS

The characteristics of the acceleration lanes studied and the major conclusions reached for each type of analysis are summarized below. Also, some additional conclusions are drawn, based on the combination of the conclusions reached from the analysis and the review of existing practice.

6.1 Operational Analysis

Acceleration Lanes for Right Turning Vehicles

The flow rates (one-directional) and the lengths of acceleration lanes studied, are as follows:

	<u>Flow Rate</u> (vph)	<u>Length</u> (ft.)
Four-Lane divided	1000	200
	1500	600
	1100	60
Two-Lane	600	200
	700	127

The merging characteristics and speeds at an acceleration lane with a length of 957 ft. was also studied. All cross roads had two lanes.

Delay Analysis:

1. Acceleration lanes appear to decrease delay for right turning vehicles from the cross road. The measured delay consisted of

delay at the stop bar and stopped delay incurred while attempting to merge. This was true for intersections in the four-lane as well as in the two-lane category.

2. Low standard acceleration lanes exhibited larger percentages of decrease in delay than the longer acceleration lanes in the four-lane categories. In the two-lane category, the high standard acceleration lane performed better.
3. It could be expected that the performance of acceleration lanes at two-lane intersections should be better than those at four-lane intersections, since vehicles at two-lane intersections cannot move over when vehicles want to merge. The results, however, do not bear this out.
4. An economic analysis indicated that acceleration lanes shorter than 200 ft. could be economically justified, if a value of \$14.80 or less were to be assigned to travel time savings of only a few seconds per vehicle. It should be noted that the cost of right of way was not included in the analysis. The only acceleration lane longer than 200 ft., i.e. one with a length of 600 ft., required a travel time value of approximately \$115.00 per hour.

Merging Analysis

1. On four lane-highways, merging appeared to be comfortable for acceleration lanes longer than 600 ft. (for a through traffic flow

rate range of 500 - 2500 vph in one direction). Merging did not appear to be comfortable for acceleration lanes shorter than 200 ft. (for a through traffic flow rate range of 500 - 1500 vph in one direction).

2. Acceleration lanes on two-lane highways should be longer than 200 ft. for comfortable merging (for a through traffic flow rate range of 500 - 1500 vph in one direction).
3. Acceleration lanes should be longer on two-lane highways than on four-lane highways for comparable through traffic flow rates (500 to 1500 vph range).
4. Vehicles often tended to use the shorter lanes (less than 200 ft.) as a refuge to wait for a gap, while the longer lanes are utilized to accelerate in order to merge with the through traffic.
5. A large percentage of vehicles did not come to a complete stop at the stop bar when turning right. Instead, they executed what may be termed a "rolling stop". This phenomenon may warrant further investigation.

Speeds:

1. The presence of acceleration lanes do not appear to affect the average speed upstream and downstream from the intersection.

2. Longer acceleration lanes do not appear to lead to significantly lower differences between through and merging speeds. It may be that the acceleration lanes are not long enough to allow for adequate acceleration or that drivers do not know how to use them.
3. The differences between through and merging speeds appear to be lower for two-lane highways than at four-lane highways.

Level of Service Analysis:

The addition of an acceleration lane for right turning vehicles significantly improves operations for this movement but does not have as great an impact on the shared lane (right turns, left turns and through movements shared this approach) on the cross road nor on the rest of the intersection.

Acceleration Lanes for Left Turning Vehicles

The lengths of (excluding tapers) and two-directional flow rates at the acceleration lanes studied are as follows:

	<u>Flow Rate</u> (vph)	<u>Length</u> (ft.)
Four-Lane divided	1500	600
	2600	118
	4600	225
	3500	132
Two-Lane	2200	216
	1600	76

Delay Analysis:

1. Acceleration lanes for left turning vehicles do not lead to a significant decrease in delay and is therefore, from this point of view, not economically justified.
2. The high standard acceleration lanes performed better than the low standard lanes in the four-lane category. In the two-lane category, circumstances at the sites precluded a clear conclusion.
3. Greater benefits could be expected from an acceleration lane in the four-lane narrow median category than in the wide median category, since vehicles cannot use the median as a refuge. The results, however, do not bear this out.
4. Notwithstanding the expectation that greater benefits could be expected from acceleration lanes on two-lane highways than on four-lane highways (since vehicles cannot give way to merging vehicles), the results did not confirm this expectation.

Merging Analysis:

1. For left turning vehicles on four-lane wide median highways, the required length of acceleration lanes for comfortable merging appears to be longer than for a right turn acceleration lane. It appears that the length required for comfortable merging may be longer than 600 ft. when the two-directional flow rate exceeded

1500 vph. Vehicles tended to use the short acceleration lanes (less than 118 ft.) as a refuge.

2. In the four-lane narrow median category, the acceleration lane should be longer than 225 ft. to allow for comfortable merging at flow rates higher than 3500 vph. Vehicles used lanes shorter than 132 ft. as a refuge.
3. The length of acceleration lanes for left turning vehicles on two-lane highways should be longer than 216 ft. for comfortable merging above two-directional flow rates of 1500 vph. The acceleration lane with a length of 76 ft. was used as a refuge.
4. Vehicles used the short left turn acceleration lanes more often as a refuge to wait for a gap than in the case of right turns.

Speeds:

1. The presence of acceleration lanes do not appear to affect the average speed upstream and downstream from the intersection.
2. Merging speeds from short acceleration lanes for left turning vehicles appear to be lower than at other acceleration lanes.
3. Longer acceleration lanes do not appear to lead to significantly lower differences between through and merging speeds. It may be that the acceleration lanes are not long enough to allow for

adequate acceleration or that drivers do not know how to use them.

4. The differences between through and merging speeds appear to be lower for two-lane highways than at four-lane highways.

General Observations:

The following general observations were made regarding traffic operations related to turning movements from the cross roads:

1. Some drivers caused long delays at intersections with acceleration lanes due to the fact that they did not appear to know how to use the acceleration lanes.
2. At high traffic flow rates, right turning vehicles tended to use the full length of the acceleration lane before merging.
3. At sites without acceleration lanes, vehicles sometimes used the shoulder as an acceleration lane when there were few suitable gaps available for merging from a stopped position. Some vehicles travelled up to 200 ft. on the shoulder before merging.
4. Vehicles making left turns onto highways, without acceleration lanes, very often stop in the median before merging with the through traffic. This was particularly true when the through traffic flow rate was high.

6.2 Safety Analysis

The characteristics of the acceleration lanes studied were as follows:

	<u>ADT</u> (through traffic)	<u>Length</u> (ft.)
Right Turn Lanes		
Four-Lane Highways	19,900 to 64,300	60 to 957
Two-Lane Highways	5,000 to 30,800	58 to 300
Left Turn Lanes		
Four-Lane Highways	14,500 to 64,300	60 to 600
Two-Lane Highways	19,700 to 30,900	65 to 216

1. Left turn acceleration lanes yielded slightly higher decreases in accident rates and costs than right turn acceleration lanes.
2. Based on absolute cost (as opposed to cost per million vehicles), right turn acceleration lanes performed better on two-lane roads than on four-lane roads.
3. For left turns, acceleration lanes on two-lane highways performed better than on four-lane highways, followed by the four-lane narrow median category.
4. The greatest reduction in accident rates and costs occurred for lengths up to approximately 200 ft. Benefits were also found for lengths of 957 ft. for right turns and up to 600 ft. for left turns. It may also be speculated that benefits could be obtained for lengths greater than the latter.

5. It appears that the acceleration lanes studied are economically justified, given a reasonable assumption for discount rate* and service life. A real (excluding inflation) discount rate of five percent was used. The cost of right of way was not included. It should be noted that the combination of through traffic and turning traffic volume should be a factor, but these data were not available for all the sites studied during the safety analysis. It may be concluded, however, that the acceleration lanes would be justified for typical combinations of through and turning traffic volumes encountered at these types of intersections. The lowest combinations of through and turning traffic volumes which yielded a favorable benefit/cost ratio were as follows:

	ADT	Turning Flow Rates (vph)
Right Turn Lanes		
Four-Lane Highways	46,500	84
Two-Lane Highways	20,200	69
Left Turn Lanes		
Four-Lane Highways		
Wide Median	33,600	38
Narrow Median	55,600	17
Two-Lane Highways	30,900	49

* The discount rate is similar to the interest rate except that an interest rate is usually narrowly defined as a contractual arrangement between a borrower and a lender, whereas a discount rate represents the real change in value to a person or group as determined by their possibilities for productive use of resources.

These turning flow rates are in the same range as those used by the State of Colorado, but the through traffic volumes are higher than the Colorado guidelines (see Figure 3.2). It should be pointed out that it is possible that sites with lower through traffic volumes, than those studied, could also be economically feasible.

6.3 Additional Conclusions

From the conclusions presented above, it appears that acceleration lanes would be justified if right of way costs are not prohibitive. Regarding the appropriate length, it appears that lengths of approximately up to 200 ft. are adequate to provide a refuge for merging vehicles where they can wait for an appropriate gap. Longer lanes of up to 465 ft. for right turn lanes and up to 257 ft. for left turn lanes proved to be economically justified. To allow for comfortable merging (when vehicles would be able to accelerate to speeds close to that of the through traffic), lanes which are longer than 600 ft. would be necessary.

From the review of existing practice, it was found that the width of acceleration lanes should preferably be 12 ft. and not less than 10 ft. No evidence was found that other than existing practice for the design of the shoulder and taper should be followed.

7. GUIDELINES FOR IMPLEMENTATION

The suggested guidelines presented below were based on the foregoing analysis and conclusions. In order to arrive at guidelines, the factors which were taken into account are listed in order of priority:

- (1) Indications of potentially hazardous locations.
- (2) Economic considerations.
- (3) Observed ease of operation.

The proposed guidelines are as follows:

1. An economic analysis, similar to the analysis presented in the section on "Safety Analysis", should be carried out to determine whether the acceleration lane is feasible. The benefits should consist of the reduction of accidents. The reduction factors should correspond as closely as possible to the factors determined for each type and length of acceleration lane as well as the range of traffic flow rates, as determined in the "Safety Analysis" section. Construction, maintenance and right of way costs should be included. It should be noted, however, that other considerations, such as comfort and convenience, cannot be enumerated in the economic analysis and could also be taken into account in the evaluation of whether an acceleration lane should be considered. It was found that the acceleration lanes were feasible above the following combinations of traffic volume and turning flow rates from the cross road:

	ADT	Turning Flow Rates (vph)
Right Turn Lanes		
Four-Lane Highways	46,500	84
Two-Lane Highways	20,200	69
Left Turn Lanes		
Four-Lane Highways		
Wide Median	33,600	38
Narrow Median	55,600	17
Two-Lane Highways	30,900	49

2. As a starting point, an attempt should be made to make the full length of acceleration lane (not including the taper) longer than 600 ft. Table 3.1 can be used as a guide to determine the initial length. Grade adjustments could be made according to Table 3.2. It should be noted that acceleration lanes less than 600 ft. in length have not been found to allow for comfortable merging but do allow vehicles to wait for a suitable gap. It should be noted that acceleration lanes with lengths of less than 200 ft. proved to be beneficial.

3. Lane widths should preferably be 12 ft. and not less than 10 ft. Current practice for the design of shoulders and tapers should be followed.

8. RECOMMENDATIONS

Recommendations for future study are as follows:

1. In view of the fact that it was found that comfortable merging does not occur for the range of lengths of acceleration lanes studied, it is recommended that a safety analysis be conducted to determine the economic feasibility of longer acceleration lanes should they be constructed.
2. Additional research should be considered to determine the minimum turning flow rates necessary to justify implementation of acceleration lanes.
3. Consideration should be given to the phenomenon of vehicles executing a "rolling stop" when turning right into an acceleration lane.

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APPENDIX A - Example Intersection Layouts Obtained from New Hampshire

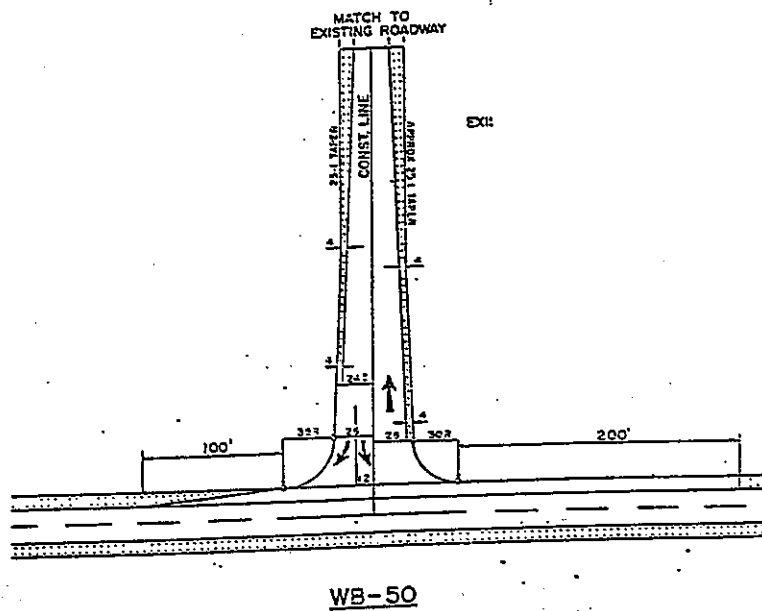


FIG. 1-A

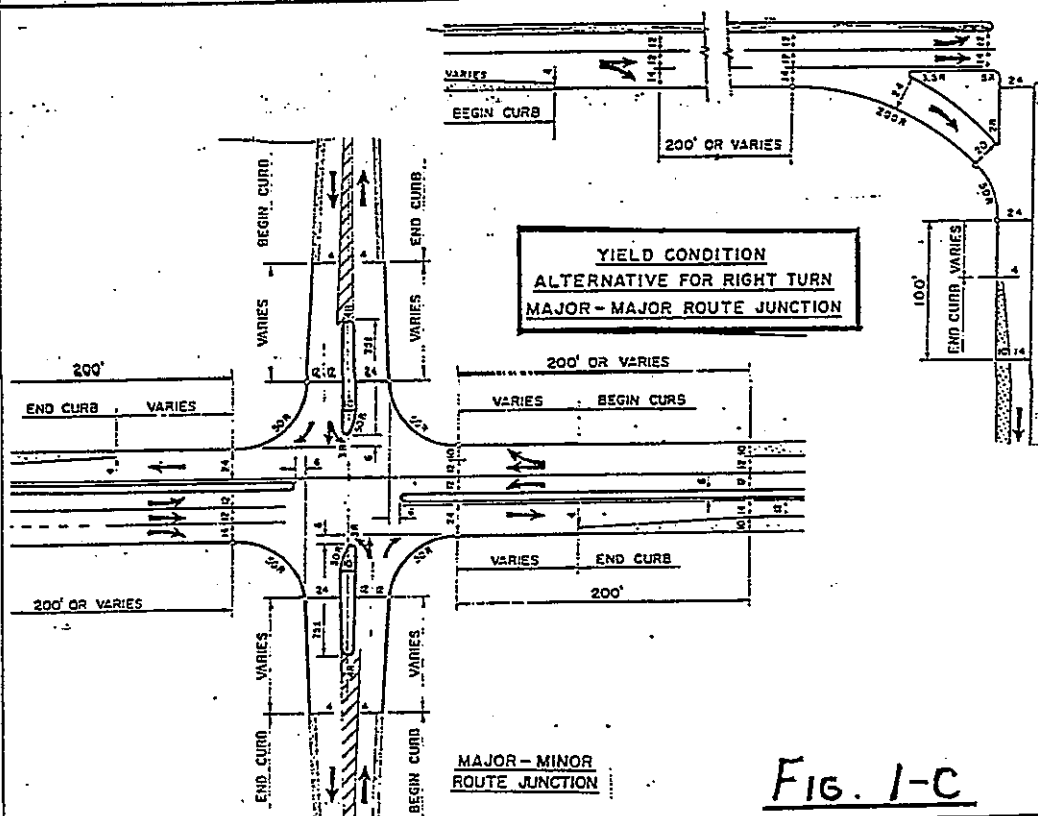


FIG. 1-C

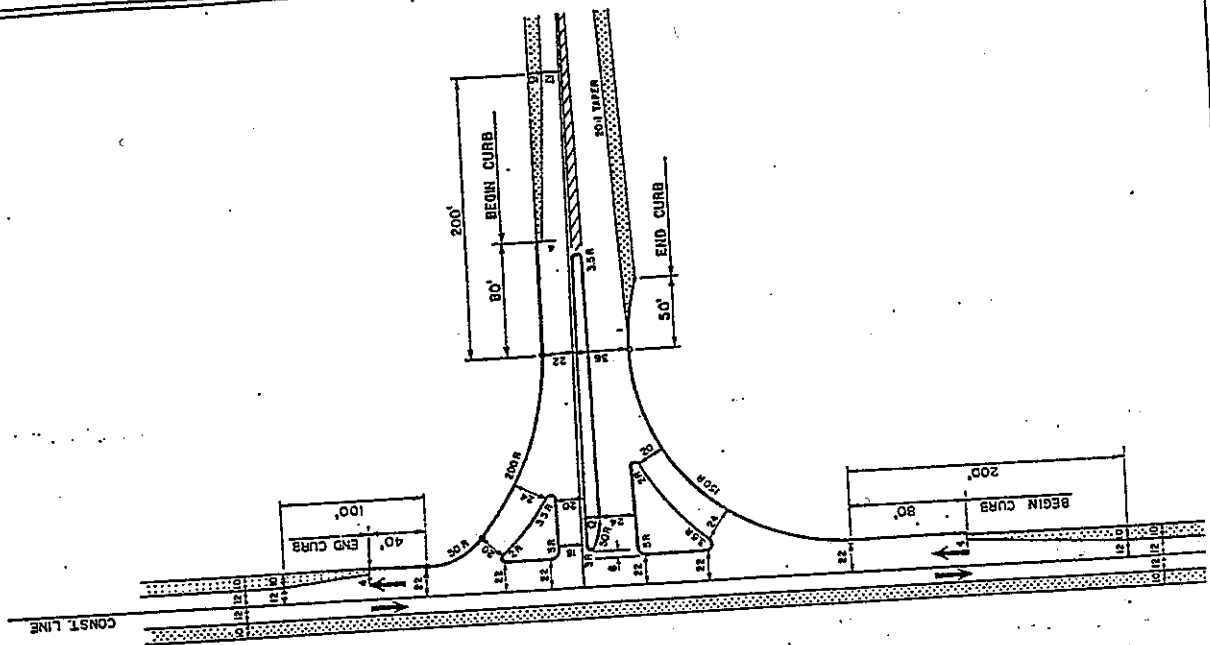


Fig. 1-B

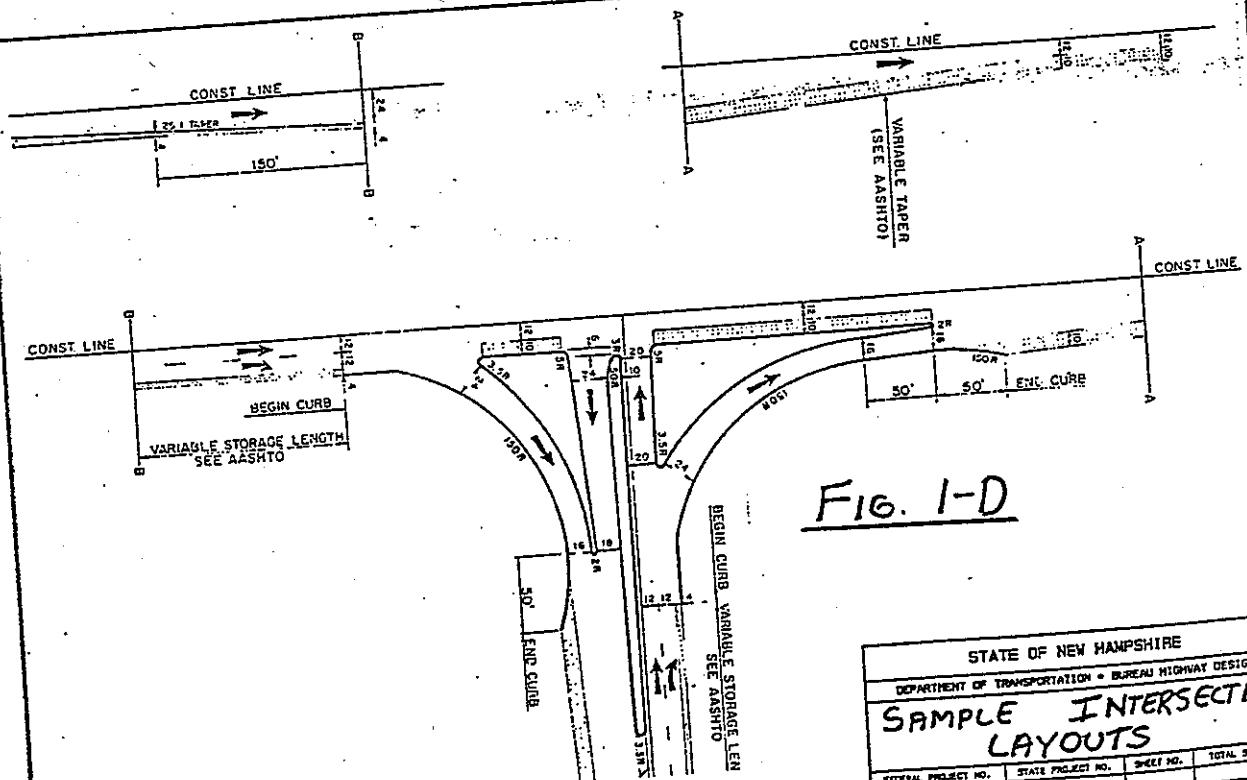


Fig. 1-D

STATE OF NEW HAMPSHIRE			
DEPARTMENT OF TRANSPORTATION • BUREAU HIGHWAY DESIGN			
SAMPLE INTERSECTION LAYOUTS			
FEDERAL PROJECT NO.	STATE PROJECT NO.	SHEET NO.	TOTAL SHEETS

APPENDIX B - Notes on Level of Service Analysis

This appendix contains a brief explanation of the results of the Level of Service (LOS) analysis of section 4.7.

The Highway Capacity Manual (HCM) method for determining the level of service of unsignalized intersections, found in chapter 10 of the HCM, uses a numbering system for turning movements. For a typical four-leg intersection, there are three movements per leg for a total of twelve movements, indicated by v1 through v12. The movements are numbered counter-clockwise starting with the left turns from the major road. Following this pattern, the right turns from the minor road would be numbered v9 and v12. These numbers are not, for the purposes of this report, dependent on the cardinal directions (North, South, East, or West), but rather dependent on the origin of the movement, i.e., main or minor road.

The results of the analysis are shown in Figures B1 through B14. The reserve capacities of the right turn movement or the shared lane on the minor road corresponding to volume increases from the base volumes, are presented. The base volumes for each intersection are shown at the bottom of each figure. In the case of T-intersections, all movements would not be present and the corresponding volume would not be listed or listed as zero.

For example, "Base w/o (rt)" indicates the base condition volumes which are being tested (Base), without a right-turn acceleration lane for the

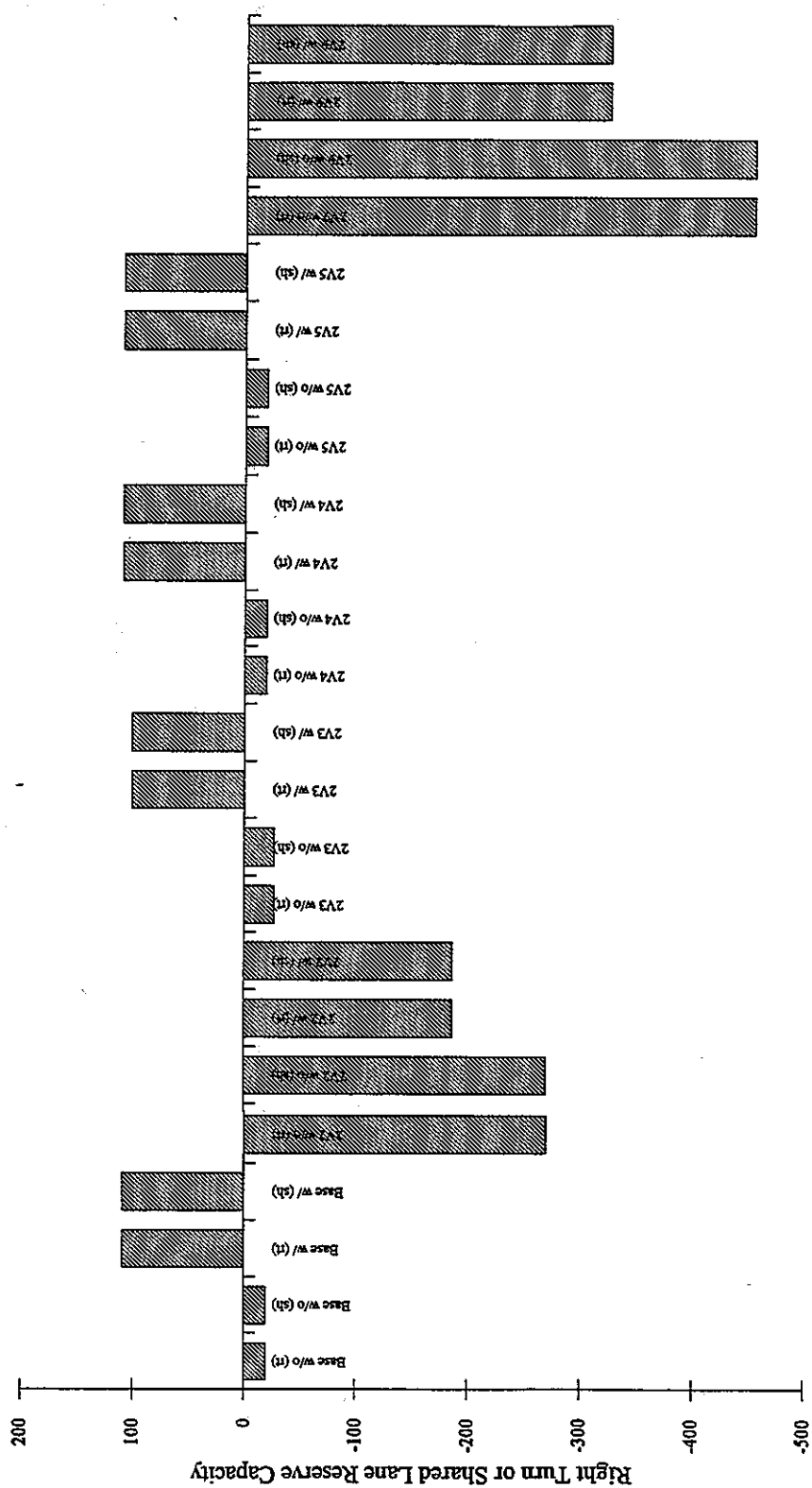
minor street (w/o), and the resulting reserve capacity is for the right turn movement (rt). It was determined that for a four-leg intersection, the reserve capacity of one right turn movement from the minor road was independent of the reserve capacity of the right turning movement from the minor street on the opposite approach. Therefore, the without acceleration lane calculations for both opposing approaches were performed simultaneously. Similarly, the with acceleration lane calculations were performed simultaneously for both opposing approaches.

The "w/" symbol indicates that acceleration lanes are present for both right turn movements from the minor road opposing legs, where both legs exist or just the one in the case of T-intersections. The symbol (sh) indicates that the displayed reserve capacity is for the shared lane approach, not the right turn movement alone. The symbols other than the "Base" cases, such as "2v2", indicate the factor by which the indicated volume has been increased. As above, "2v2" indicates that v2 has been doubled. Only one subject volume is changed at a time, and this volume will be the only volume differing from its base condition at that time. Each volume case will have four reserve capacities indicated: without acceleration lane for the right turning movement ("w/o (rt)"), without acceleration lane for the shared lane ("w/o (sh)"), with acceleration lane for the right turning movement ("w/ (rt)"), with acceleration lane for the shared lane ("w/ (sh)").

The shared lane capacity can be converted into the level of service categories as shown on Table 4.21 in section 4.7 of this document.

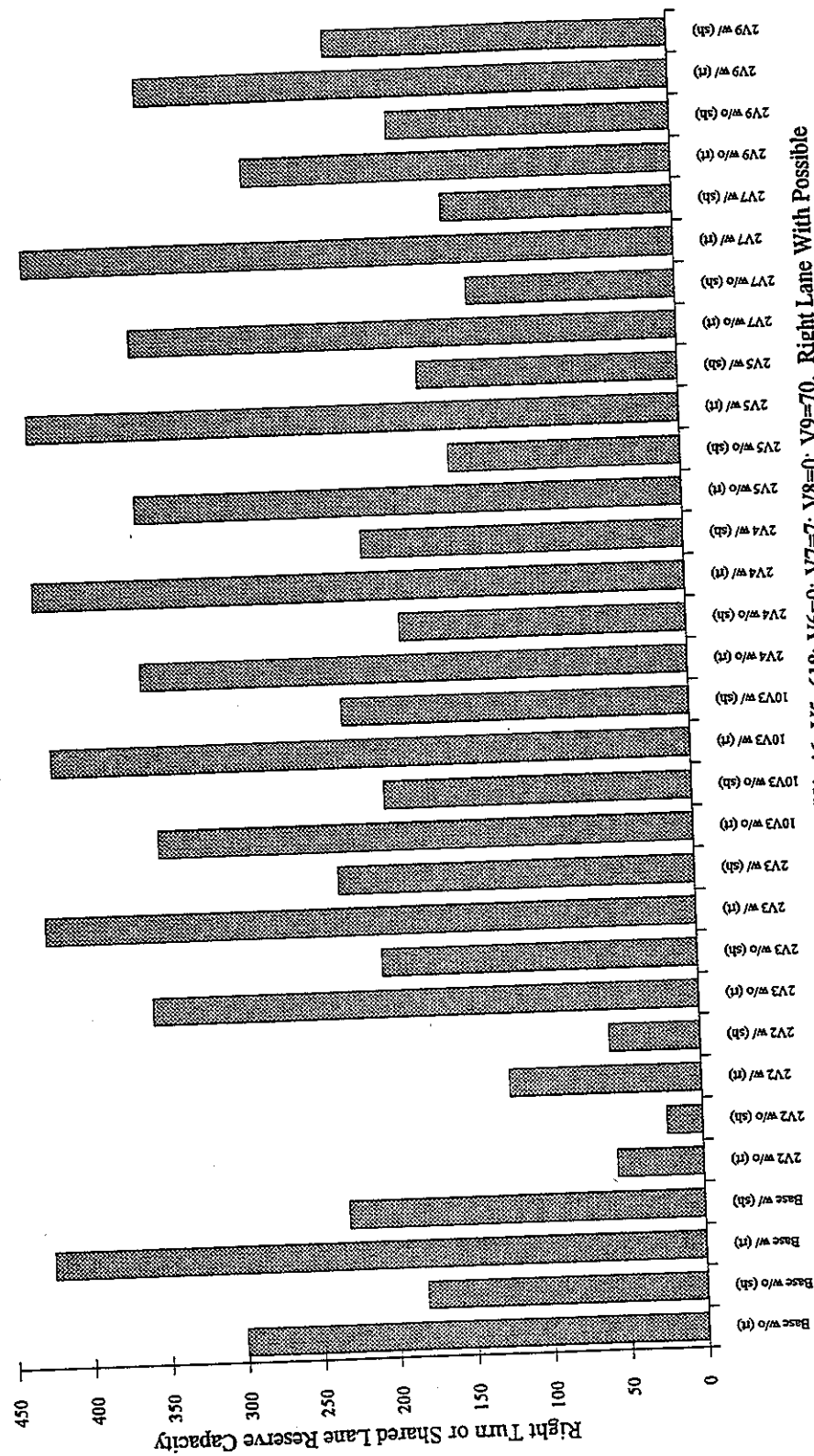
FIGURE B.1: Effect of Right Turn Acceleration Lane on Reserve Capacity

HWY 183: 5 MON-1-92.213 ("T" Intersection)



Base Case Volumes: V1=0; V2=605; V3=25; V4=421; V5=564; V6=0; V7=0; V8=0; V9=395. Right Lane With Possible Acceleration Lane=V9.

FIGURE B.2: Effect of Right Turn Acceleration Lane on Reserve Capacity
 Cuttings Wharf. 4 NAP-121-3.04 ("T" Intersection)



Base Case Volumes: V1=0; V2=681; V3=3; V4=46; V5=618; V6=0; V7=7; V8=0; V9=70. Right Lane With Possible Acceleration Lane=V9.

FIGURE B.3: Effect of Right Turn Acceleration Lane on Reserve Capacity
 Bloomfield: 4 SCL-152-14.89 ("T" Intersection)

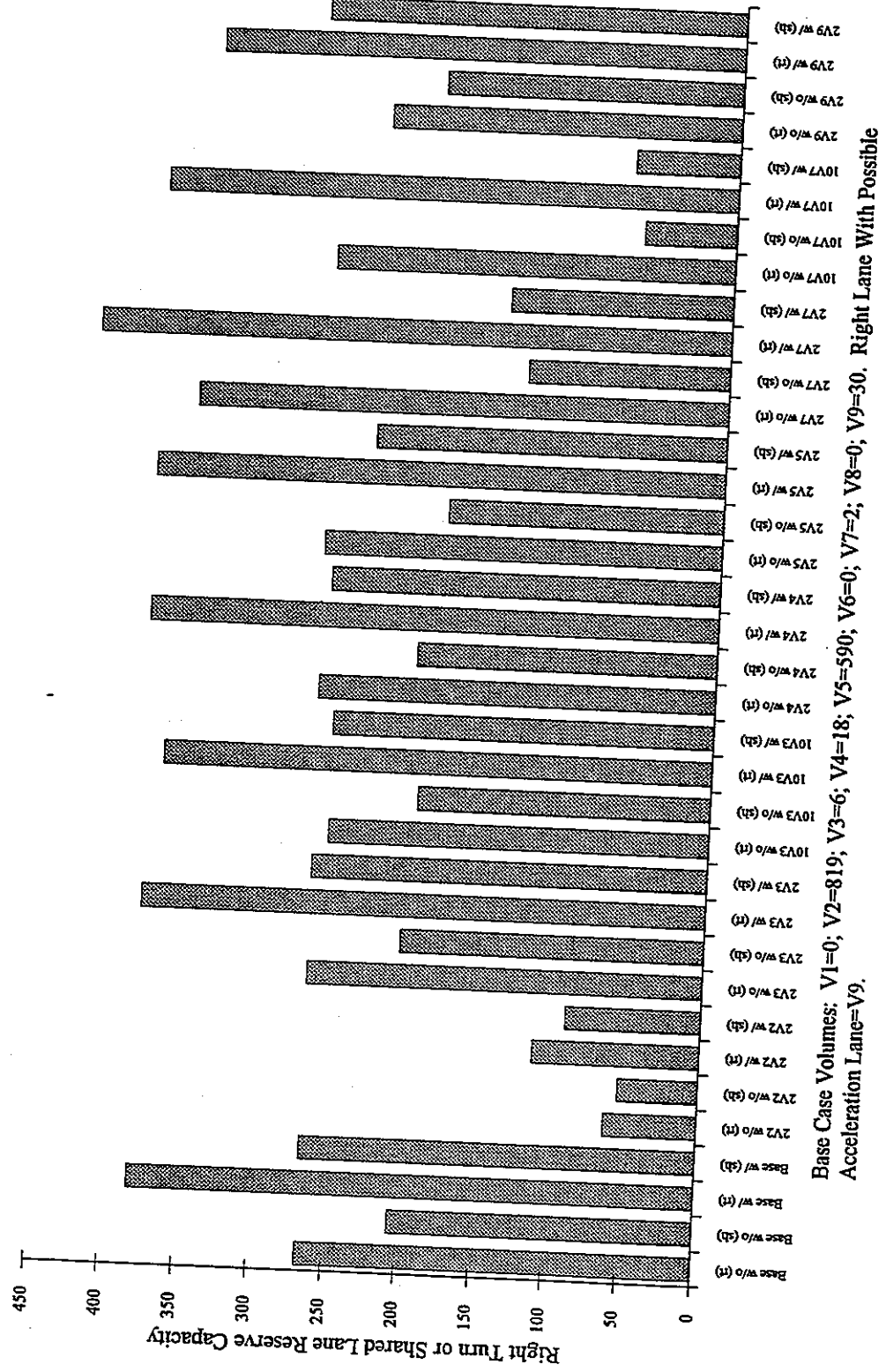
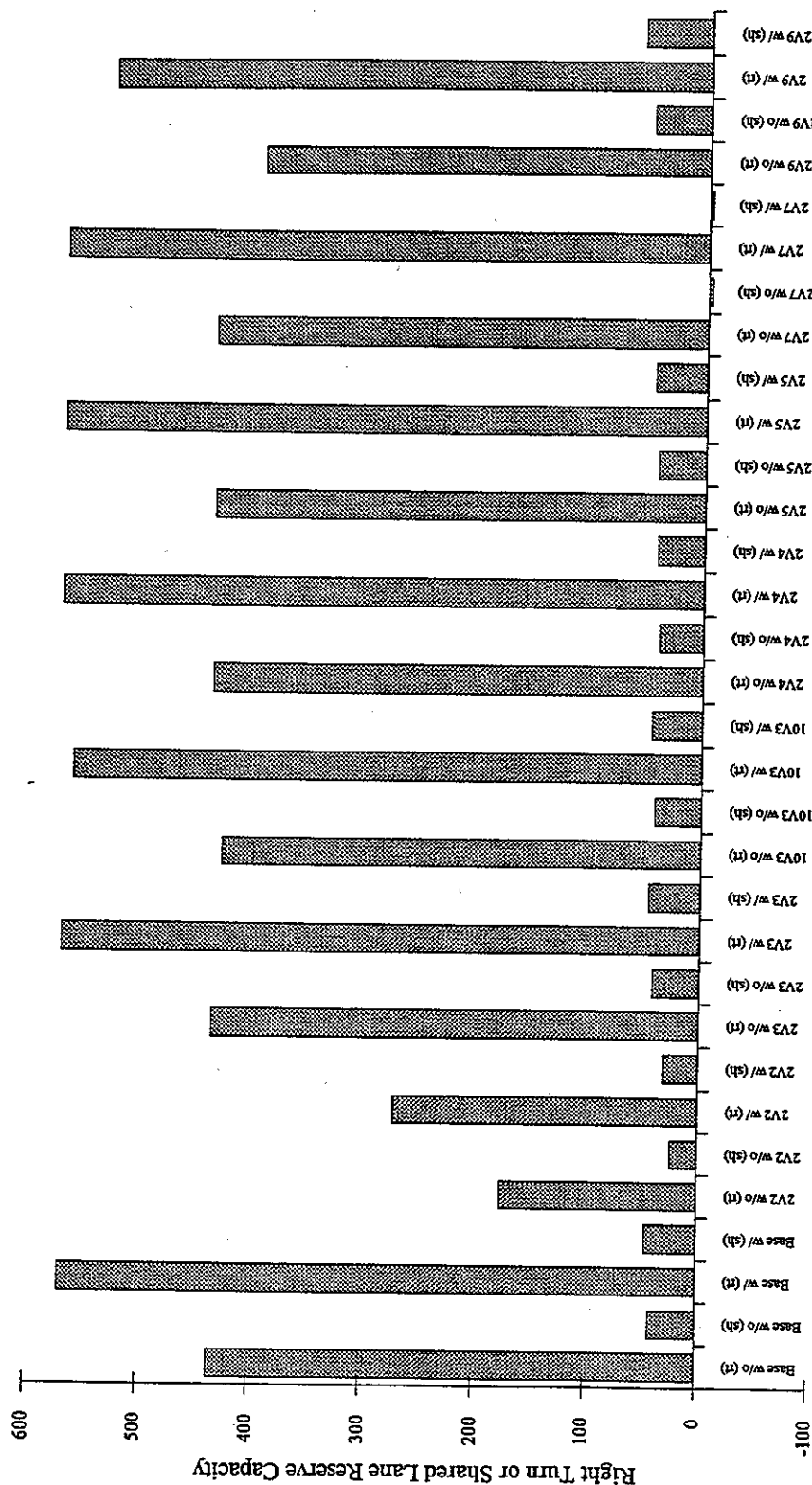


FIGURE B.4: Effect of Right Turn Acceleration Lane on Reserve Capacity

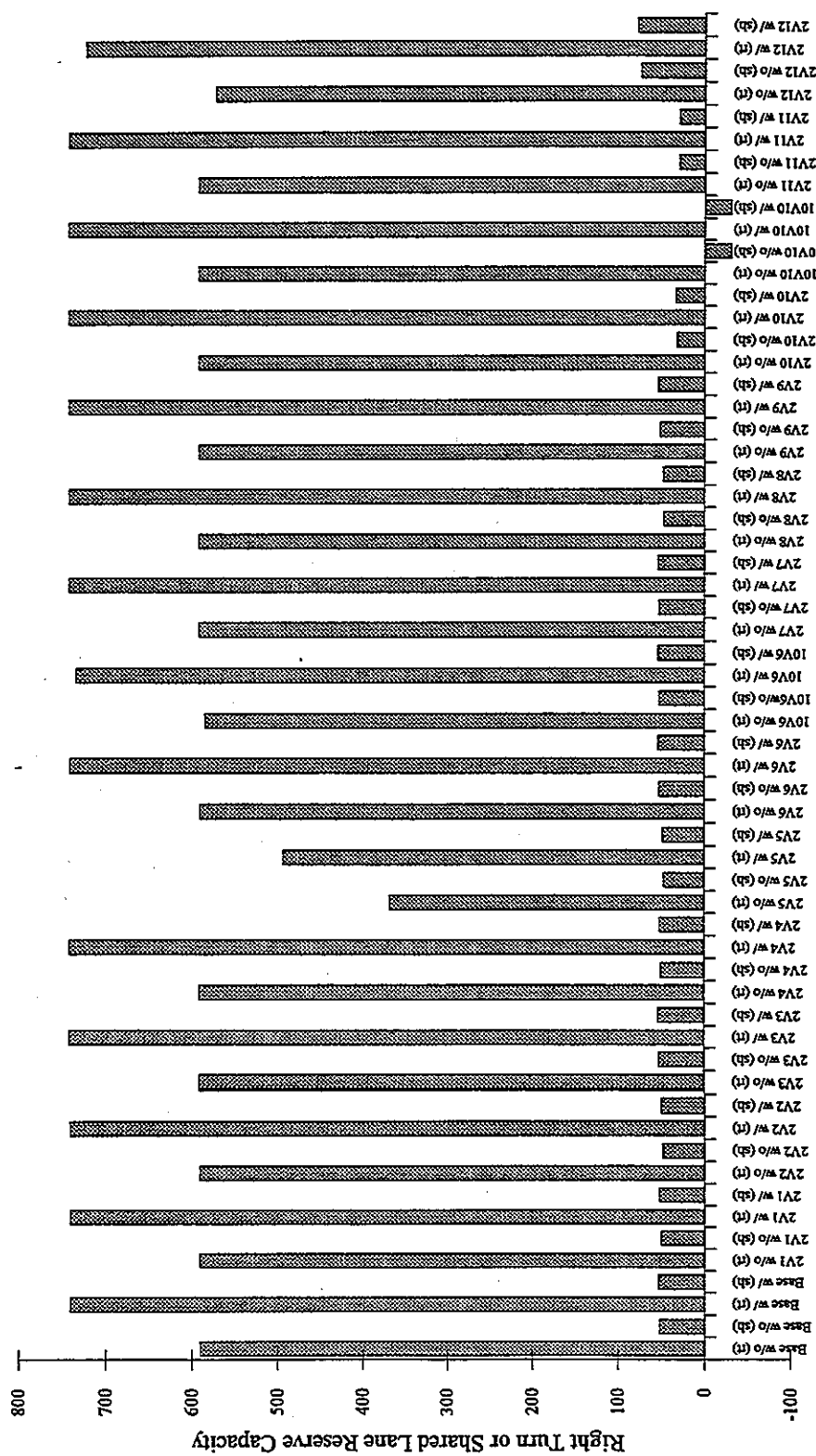
Echo Valley: 5 MON-101-98.38 ("T" Intersection)



Base Case Volumes: V1=0; V2=1044; V3=3; V4=14; V5=890; V6=0; V7=15; V8=0; V9=37. Right Lane With Possible Acceleration Lane=V9.

FIGURE B.5: Effect of Right Turn Acceleration Lane on Reserve Capacity

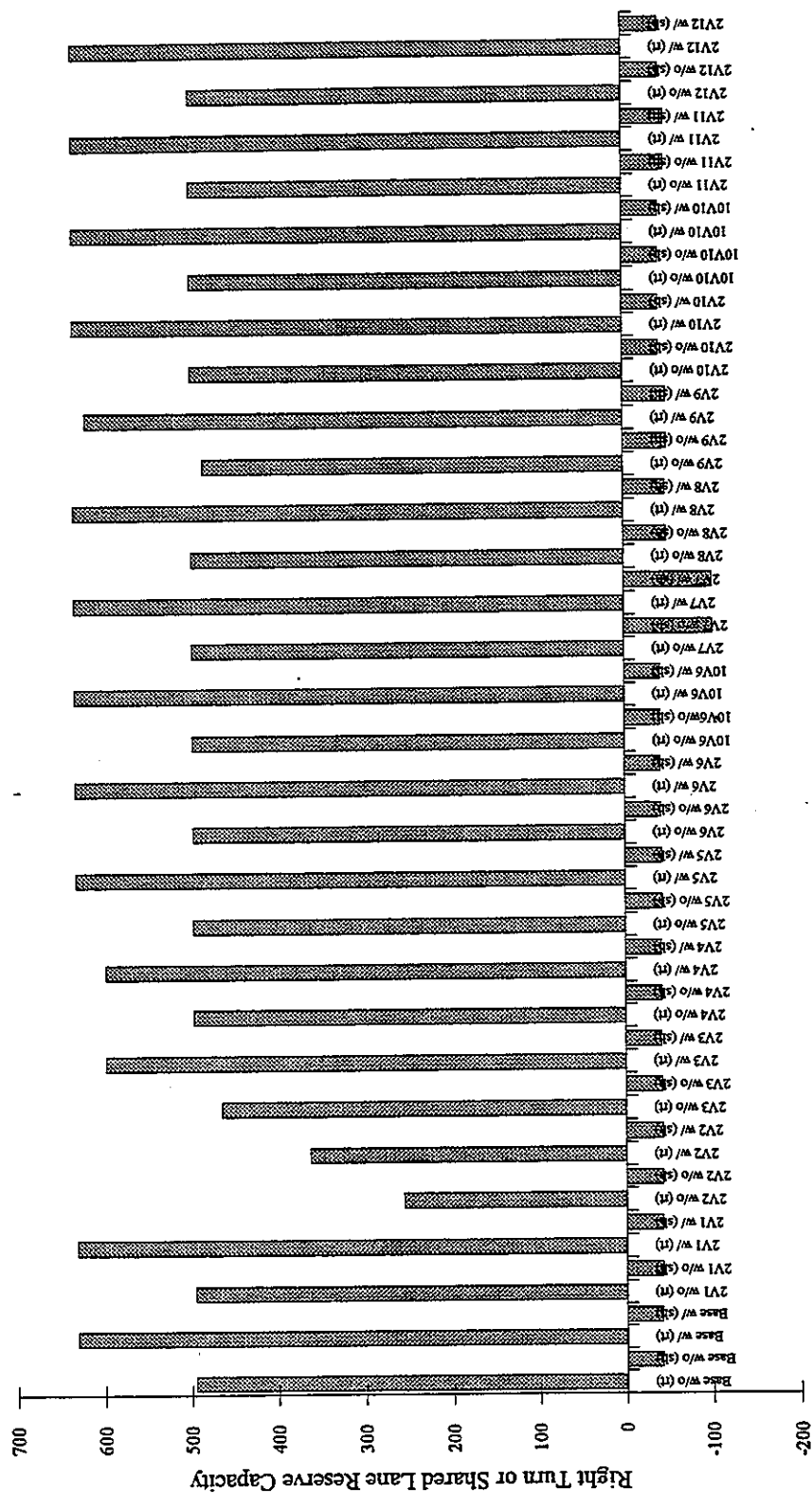
Elverta: 3 SAC-99-35.37 ("4 leg" Intersection)



Base Case Volumes: V1=14; V2=854; V3=96; V4=11; V5=665; V6=2; V7=51; V8=7; V9=13; V10=4; V11=8; V12=18.
Right Lane With Possible Acceleration Lane=V12.

FIGURE B.6: Effect of Right Turn Acceleration Lane on Reserve Capacity

Elverta: 3 SAC-99-35.37 ("4 leg" Intersection)



Base Case Volumes: V1=14; V2=854; V3=96; V4=11; V5=665; V6=2; V7=51; V8=7; V9=13; V10=4; V11=8; V12=18.
Right Lane With Possible Acceleration Lane=V9.

FIGURE B.7: Effect of Right Turn Acceleration Lane on Reserve Capacity

Spence: 5 MON-101-81.03 ("4 leg" Intersection)

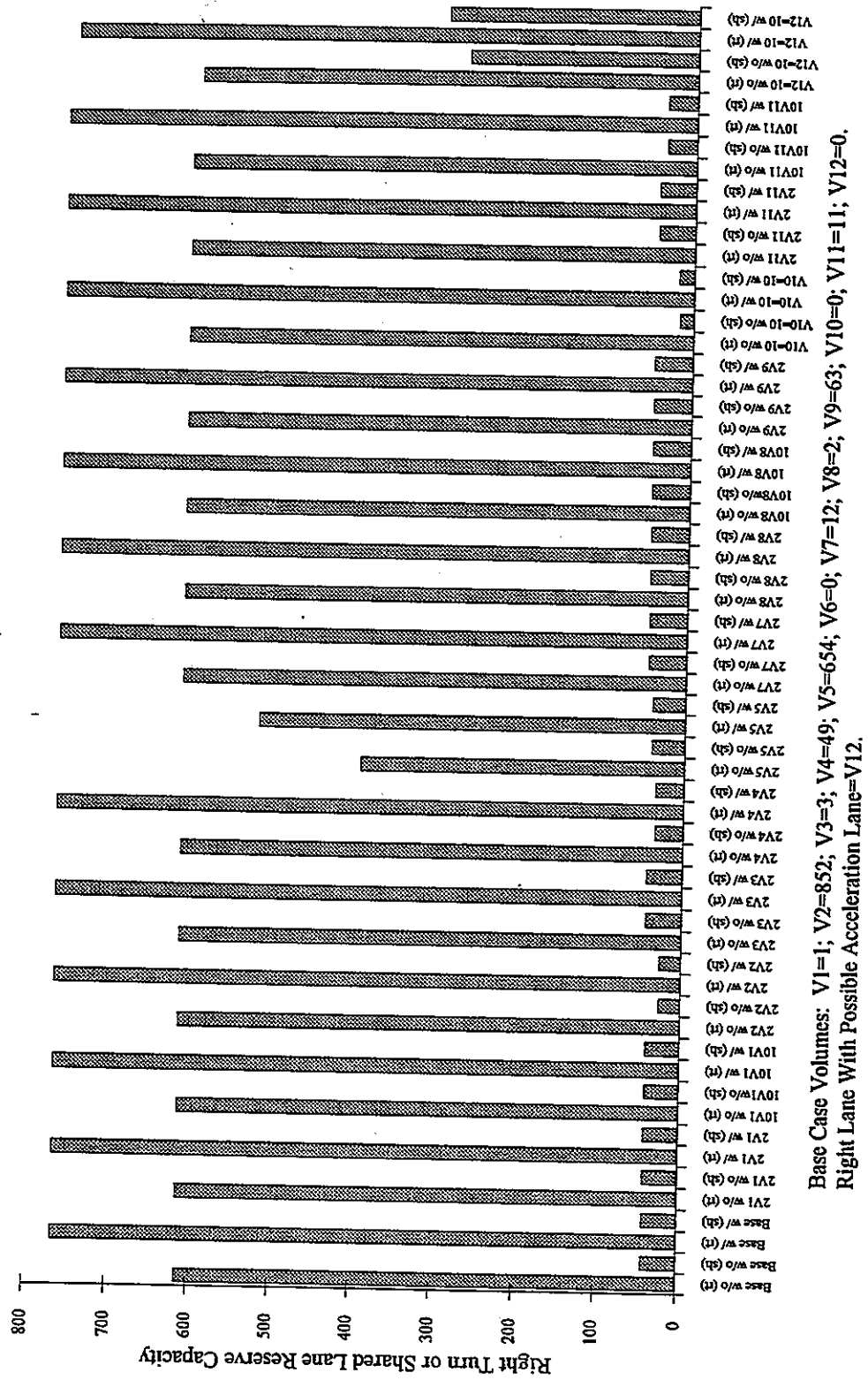


FIGURE B.8: Effect of Right Turn Acceleration Lane on Reserve Capacity

Spence: 5 MON-101-81.03 ("4 leg" Intersection)

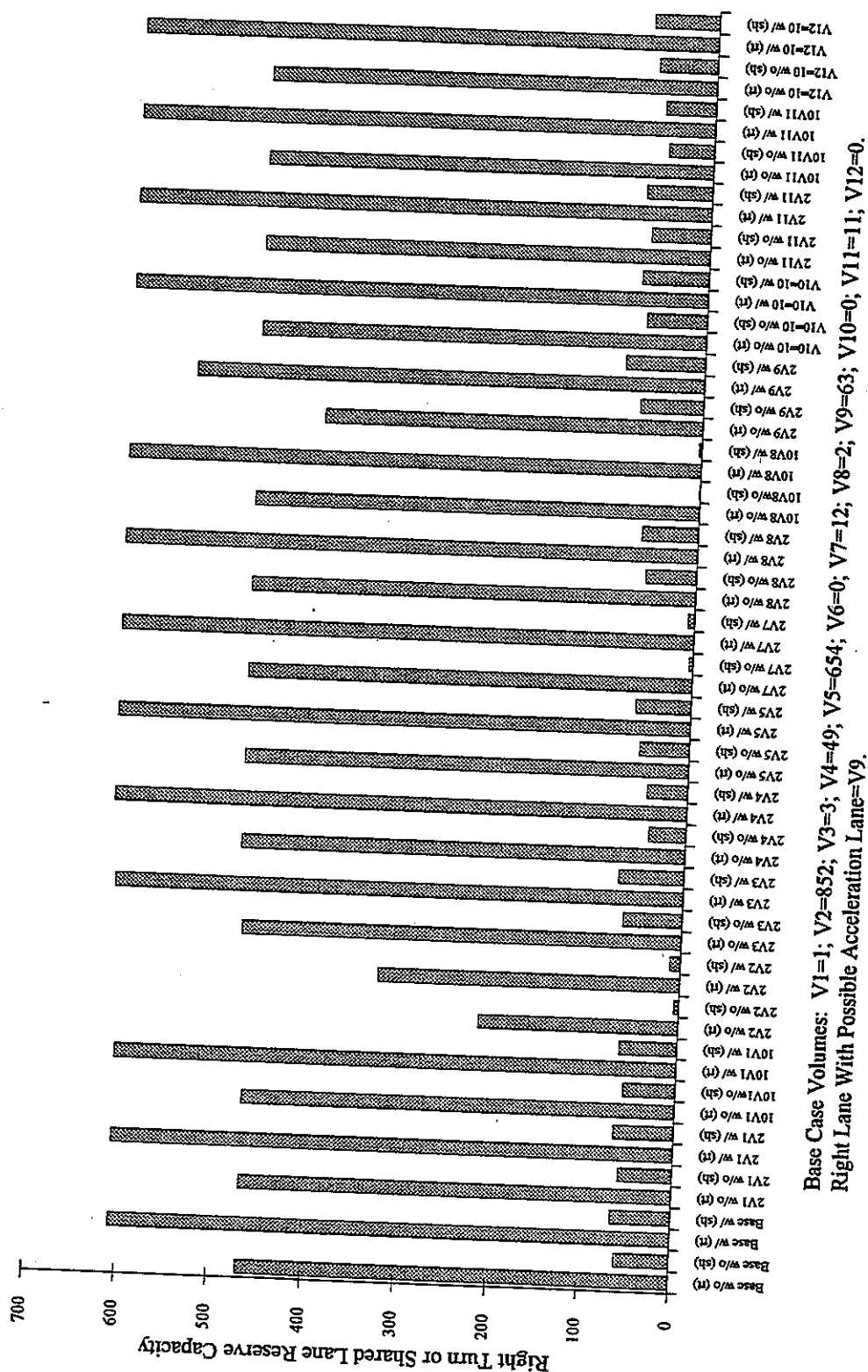
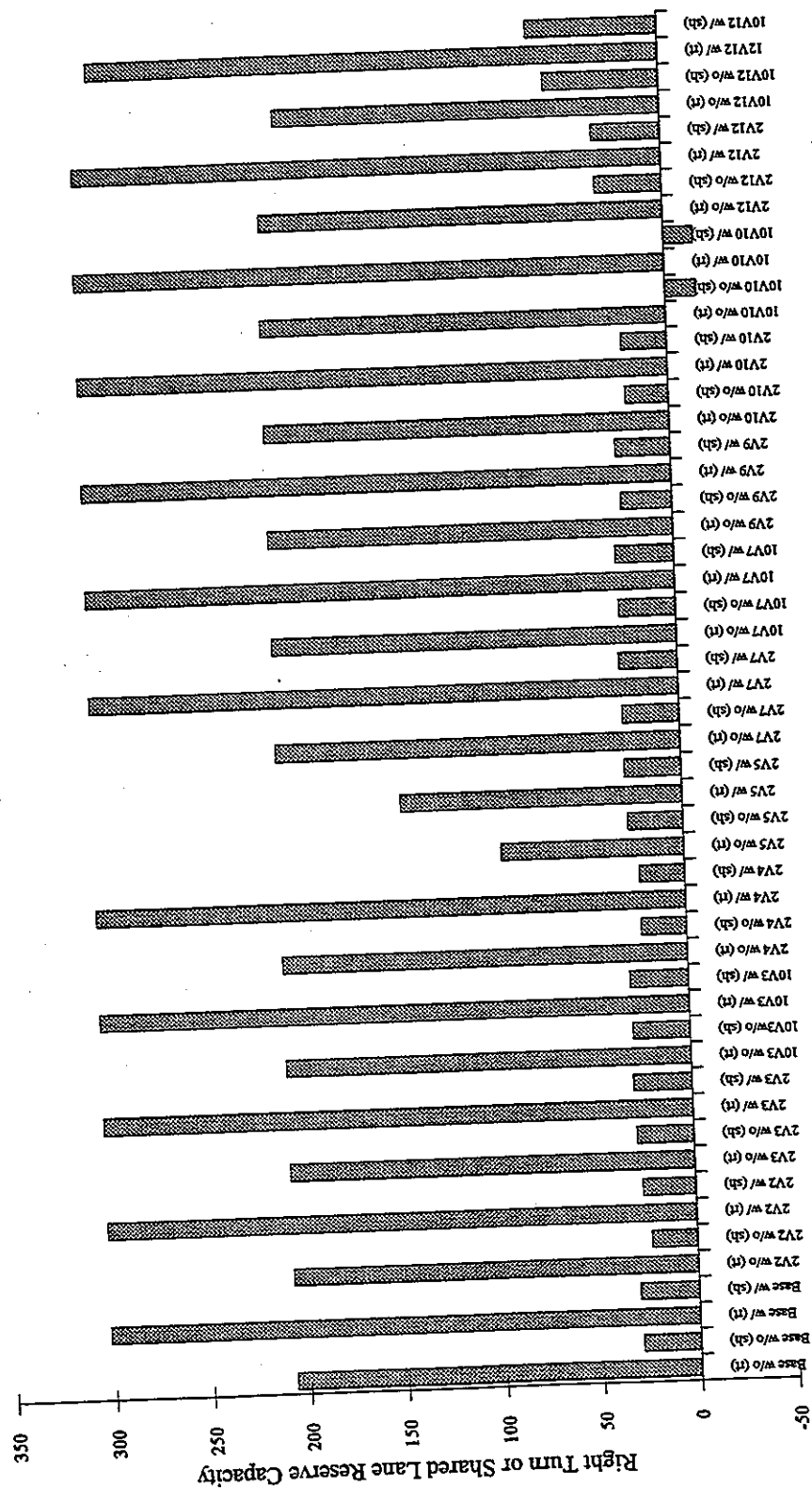


FIGURE B.9: Effect of Right Turn Acceleration Lane on Reserve Capacity

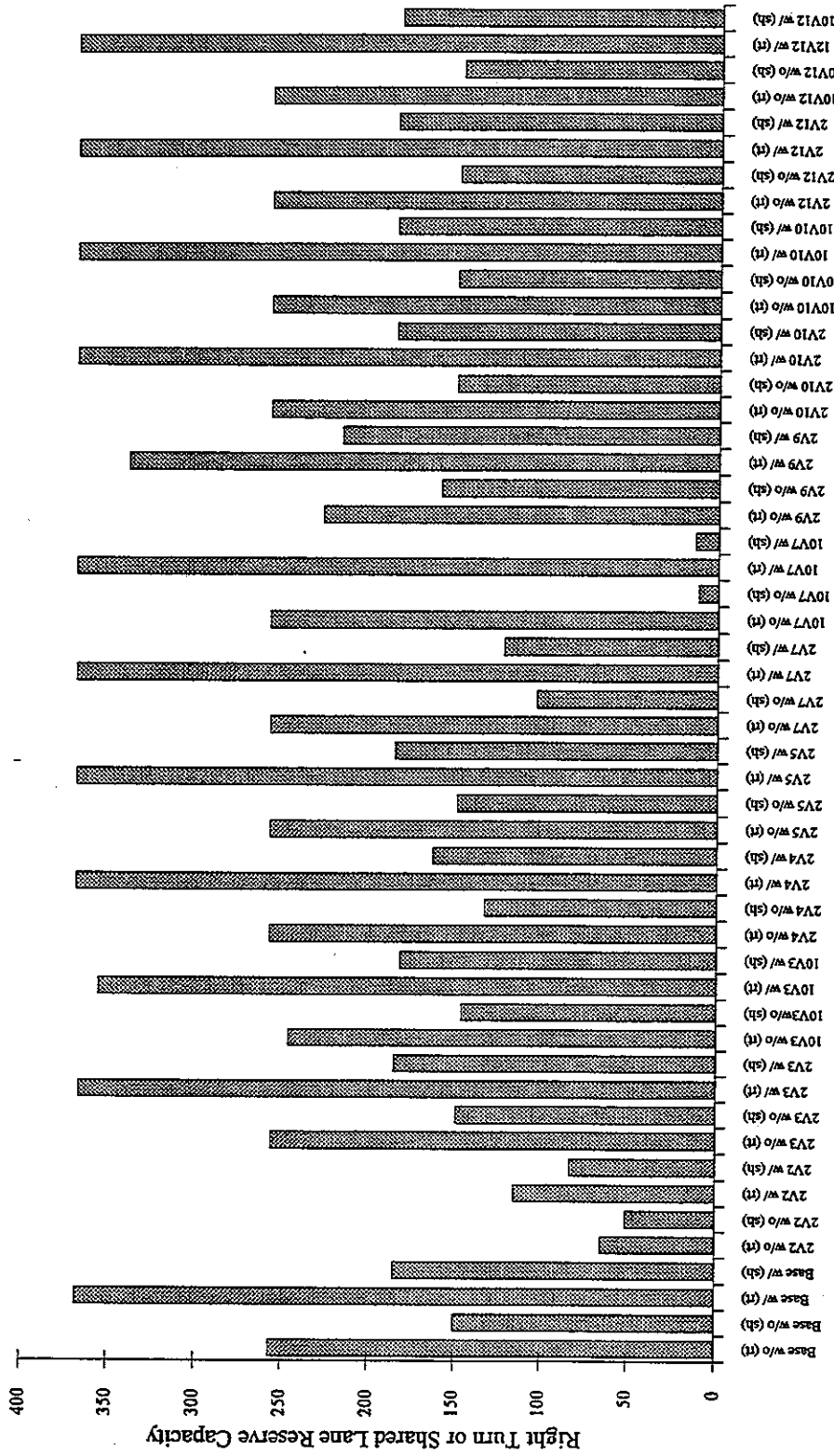
Monte-Vina: 4 SCL-17-4.62 ("4 leg" Intersection)



Base Case Volumes: V1=0; V2=1711; V3=6; V4=19; V5=2144; V6=0; V7=2; V8=0; V9=27; V10=0; V11=0; V12=0; V13=0; V14=0; V15=0; V16=0; V17=0; V18=0; V19=0; V20=0; V21=0; V22=0; V23=0; V24=0; V25=0; V26=0; V27=0; V28=0; V29=0; V30=0; V31=0; V32=0; V33=0; V34=0; V35=0; V36=0; V37=0; V38=0; V39=0; V40=0; V41=0; V42=0; V43=0; V44=0; V45=0; V46=0; V47=0; V48=0; V49=0; V50=0; V51=0; V52=0; V53=0; V54=0; V55=0; V56=0; V57=0; V58=0; V59=0; V60=0; V61=0; V62=0; V63=0; V64=0; V65=0; V66=0; V67=0; V68=0; V69=0; V70=0; V71=0; V72=0; V73=0; V74=0; V75=0; V76=0; V77=0; V78=0; V79=0; V80=0; V81=0; V82=0; V83=0; V84=0; V85=0; V86=0; V87=0; V88=0; V89=0; V90=0; V91=0; V92=0; V93=0; V94=0; V95=0; V96=0; V97=0; V98=0; V99=0; V100=0; V101=0; V102=0; V103=0; V104=0; V105=0; V106=0; V107=0; V108=0; V109=0; V110=0; V111=0; V112=0; V113=0; V114=0; V115=0; V116=0; V117=0; V118=0; V119=0; V120=0; V121=0; V122=0; V123=0; V124=0; V125=0; V126=0; V127=0; V128=0; V129=0; V130=0; V131=0; V132=0; V133=0; V134=0; V135=0; V136=0; V137=0; V138=0; V139=0; V140=0; V141=0; V142=0; V143=0; V144=0; V145=0; V146=0; V147=0; V148=0; V149=0; V150=0; V151=0; V152=0; V153=0; V154=0; V155=0; V156=0; V157=0; V158=0; V159=0; V160=0; V161=0; V162=0; V163=0; V164=0; V165=0; V166=0; V167=0; V168=0; V169=0; V170=0; V171=0; V172=0; V173=0; V174=0; V175=0; V176=0; V177=0; V178=0; V179=0; V180=0; V181=0; V182=0; V183=0; V184=0; V185=0; V186=0; V187=0; V188=0; V189=0; V190=0; V191=0; V192=0; V193=0; V194=0; V195=0; V196=0; V197=0; V198=0; V199=0; V200=0; V201=0; V202=0; V203=0; V204=0; V205=0; V206=0; V207=0; V208=0; V209=0; V210=0; V211=0; V212=0; V213=0; V214=0; V215=0; V216=0; V217=0; V218=0; V219=0; V220=0; V221=0; V222=0; V223=0; V224=0; V225=0; V226=0; V227=0; V228=0; V229=0; V230=0; V231=0; V232=0; V233=0; V234=0; V235=0; V236=0; V237=0; V238=0; V239=0; V240=0; V241=0; V242=0; V243=0; V244=0; V245=0; V246=0; V247=0; V248=0; V249=0; V250=0; V251=0; V252=0; V253=0; V254=0; V255=0; V256=0; V257=0; V258=0; V259=0; V260=0; V261=0; V262=0; V263=0; V264=0; V265=0; V266=0; V267=0; V268=0; V269=0; V270=0; V271=0; V272=0; V273=0; V274=0; V275=0; V276=0; V277=0; V278=0; V279=0; V280=0; V281=0; V282=0; V283=0; V284=0; V285=0; V286=0; V287=0; V288=0; V289=0; V290=0; V291=0; V292=0; V293=0; V294=0; V295=0; V296=0; V297=0; V298=0; V299=0; V300=0; V301=0; V302=0; V303=0; V304=0; V305=0; V306=0; V307=0; V308=0; V309=0; V310=0; V311=0; V312=0; V313=0; V314=0; V315=0; V316=0; V317=0; V318=0; V319=0; V320=0; V321=0; V322=0; V323=0; V324=0; V325=0; V326=0; V327=0; V328=0; V329=0; V330=0; V331=0; V332=0; V333=0; V334=0; V335=0; V336=0; V337=0; V338=0; V339=0; V340=0; V341=0; V342=0; V343=0; V344=0; V345=0; V346=0; V347=0; V348=0; V349=0; V350=0; V351=0; V352=0; V353=0; V354=0; V355=0; V356=0; V357=0; V358=0; V359=0; V360=0; V361=0; V362=0; V363=0; V364=0; V365=0; V366=0; V367=0; V368=0; V369=0; V370=0; V371=0; V372=0; V373=0; V374=0; V375=0; V376=0; V377=0; V378=0; V379=0; V380=0; V381=0; V382=0; V383=0; V384=0; V385=0; V386=0; V387=0; V388=0; V389=0; V390=0; V391=0; V392=0; V393=0; V394=0; V395=0; V396=0; V397=0; V398=0; V399=0; V400=0; V401=0; V402=0; V403=0; V404=0; V405=0; V406=0; V407=0; V408=0; V409=0; V410=0; V411=0; V412=0; V413=0; V414=0; V415=0; V416=0; V417=0; V418=0; V419=0; V420=0; V421=0; V422=0; V423=0; V424=0; V425=0; V426=0; V427=0; V428=0; V429=0; V430=0; V431=0; V432=0; V433=0; V434=0; V435=0; V436=0; V437=0; V438=0; V439=0; V440=0; V441=0; V442=0; V443=0; V444=0; V445=0; V446=0; V447=0; V448=0; V449=0; V450=0; V451=0; V452=0; V453=0; V454=0; V455=0; V456=0; V457=0; V458=0; V459=0; V460=0; V461=0; V462=0; V463=0; V464=0; V465=0; V466=0; V467=0; V468=0; V469=0; V470=0; V471=0; V472=0; V473=0; V474=0; V475=0; V476=0; V477=0; V478=0; V479=0; V480=0; V481=0; V482=0; V483=0; V484=0; V485=0; V486=0; V487=0; V488=0; V489=0; V490=0; V491=0; V492=0; V493=0; V494=0; V495=0; V496=0; V497=0; V498=0; V499=0; V500=0; V501=0; V502=0; V503=0; V504=0; V505=0; V506=0; V507=0; V508=0; V509=0; V510=0; V511=0; V512=0; V513=0; V514=0; V515=0; V516=0; V517=0; V518=0; V519=0; V520=0; V521=0; V522=0; V523=0; V524=0; V525=0; V526=0; V527=0; V528=0; V529=0; V530=0; V531=0; V532=0; V533=0; V534=0; V535=0; V536=0; V537=0; V538=0; V539=0; V540=0; V541=0; V542=0; V543=0; V544=0; V545=0; V546=0; V547=0; V548=0; V549=0; V550=0; V551=0; V552=0; V553=0; V554=0; V555=0; V556=0; V557=0; V558=0; V559=0; V560=0; V561=0; V562=0; V563=0; V564=0; V565=0; V566=0; V567=0; V568=0; V569=0; V570=0; V571=0; V572=0; V573=0; V574=0; V575=0; V576=0; V577=0; V578=0; V579=0; V580=0; V581=0; V582=0; V583=0; V584=0; V585=0; V586=0; V587=0; V588=0; V589=0; V590=0; V591=0; V592=0; V593=0; V594=0; V595=0; V596=0; V597=0; V598=0; V59

FIGURE B.10: Effect of Right Turn Acceleration Lane on Reserve Capacity

Monte-Vina: 4 SCL-17-4.62 ("4 leg" Intersection)

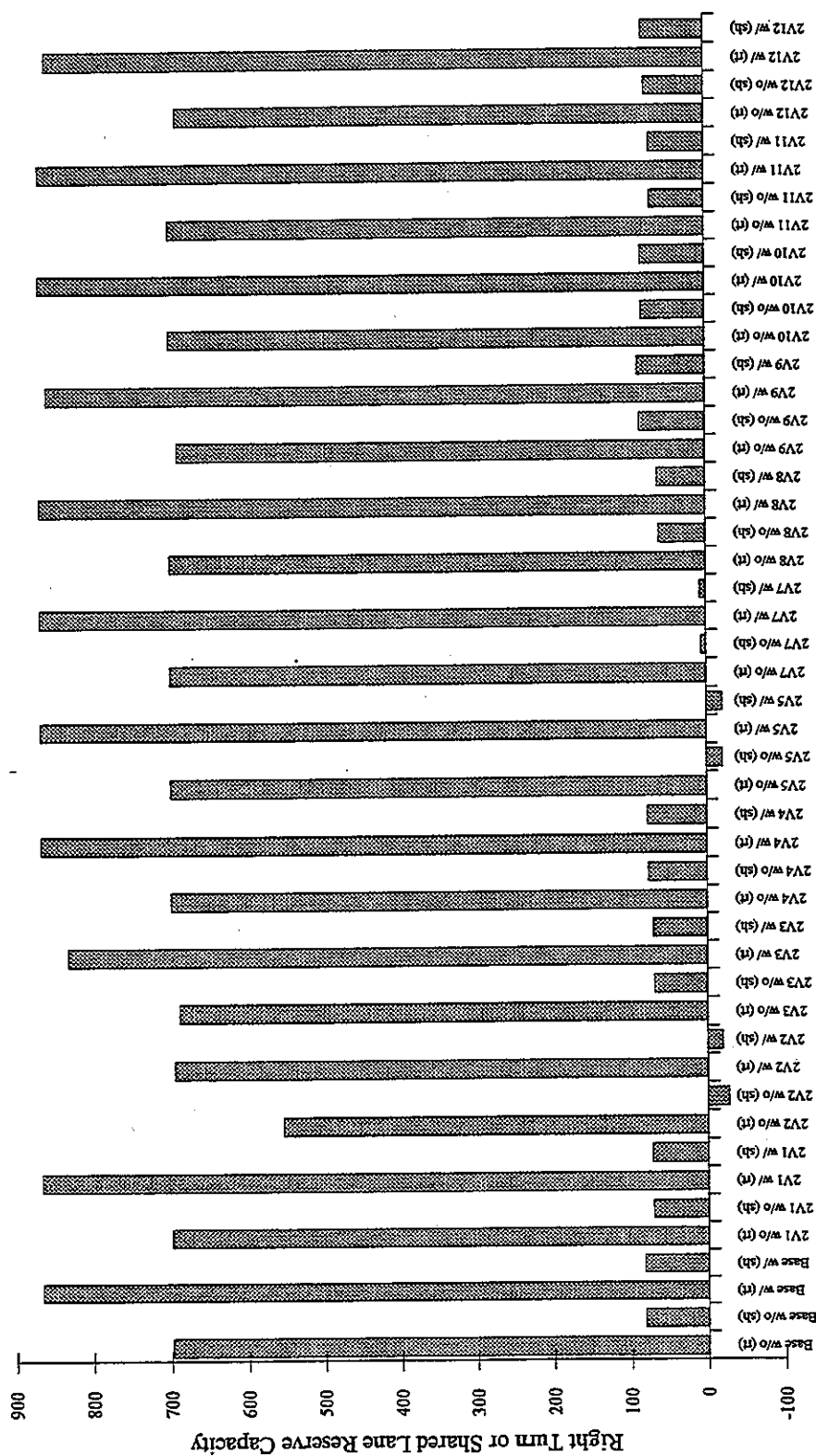


Base Case Volumes: V1=0; V2=1711; V3=6; V4=19; V5=2144; V6=0; V7=2; V8=0; V9=27; V10=4; V11=0; V12=1.
Right Lane With Possible Acceleration Lane=V9.

Base Case Volumes: V1=21; V2=360; V3=69; V4=10; V5=362; V6=0; V7=59; V8=23; V9=9; V10=8; V11=14; V12=19.
Right Lane With Possible Acceleration Lane=V12.

FIGURE B.12: Effect of Right Turn Acceleration Lane on Reserve Capacity

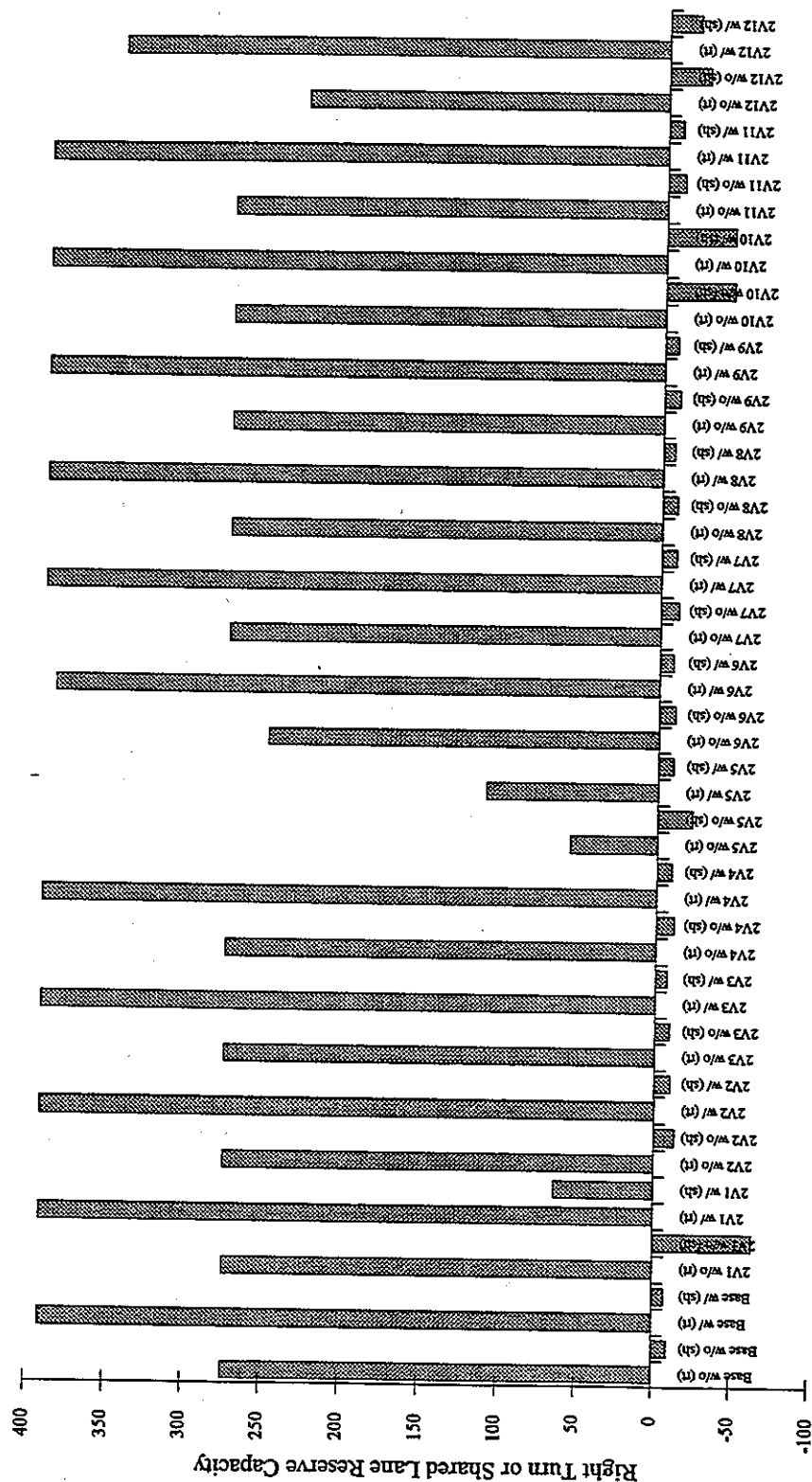
McCloskey: 5 SBT-156-11.94 ("4 leg" Intersection)



Base Case Volumes: V1=21; V2=360; V3=69; V4=10; V5=362; V6=0; V7=59; V8=23; V9=9; V10=8; V11=14; V12=19.
Right Lane With Possible Acceleration Lane=V9.

FIGURE B.13: Effect of Right Turn Acceleration Lane on Reserve Capacity

Blackie: 5 MON-101-94.28 ("4 leg" Intersection)



Base Case Volumes: V1=55; V2=1006; V3=9; V4=14; V5=1541; V6=22; V7=22; V8=1; V9=20; V10=17; V11=1; V12=42.
Right Lane With Possible Acceleration Lane=V12.